UNIVERSITY OF ILLINOIS ENGINEERING EXPERIMENT STATION CIRCULAR NO. 74

Proceedings of the Special Symposium on the Behavior of Welded Structures

conducted by the

DEPARTMENT OF CIVIL ENGINEERING

COLLEGE OF ENGINEERING
UNIVERSITY OF ILLINOIS

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ENGINEERING EXPERIMENT STATION CIRCULAR NO. 74

CONTENTS

1. W. Soete
2. G. M. Boyd
3. P. E. Lagasse
4. R. Gunnert
5. R. P. Newman
6. H. de Leiris
7. K. Ruhl
8. A. Lazard
9. F. Kellam
10. J. E. Stallmeyer
11. N. M. Newmark
12. A. M. Sullivan
13. M. A. Scheil
14. H. Fall
15. W. E. Magee
16. R. W. Vanderbeck
17. W. H. Munse
18. W. J. Hall
19. S. J. Britvec
20. J. M. Krafft
21. E. G. Wiles
22. W. W. Sanders
23. R. Cook
24. M. Marincek
25. J. W. Briscoe
26. E. H. Gaylord
27. L. J. Larson
28. D. K. Felbeck
29. R. W. Rumke
30. F. Yurkoski
31. A. L. Block
32. F. P. Worack

PREFACE

The Symposium presented herein was organized to provide for the staff and students at the University of Illinois, and for other interested persons, a summary of the research underway in some of the leading European laboratories on the "Behavior of Welded Structures." Eight internationally known engineers and scientists, all of whom are actively engaged in the activities of the International Institute of Welding, were invited to describe at the Symposium the research being conducted in their countries or in the commissions for the IIW.

The International Institute of Welding was organized in 1948 to promote the development of welding. Since 1956 the members of the Symposium Committee, members of the American Council of the IIW, have taken an active part in the Institute's activities. It was through this association that it was possible to assemble the outstanding panel of speakers provided by this Symposium. Our authors are representatives of European delegations associated with the activities of three of the fifteen commissions of the IIW, namely, Commission IX - Behavior of Metals Subjected to Loading, Commission X - Residual Stresses and Stress Relieving, and Commission XIII - Fatigue Testing.

The Symposium was made possible through the Ford Foundation's Intramural Program at the University of Illinois, through the generosity of our distinguished speakers who so graciously consented to take part in the program, and through the efforts of a number of people on the staff of the Civil Engineering Department who have worked together in the planning and presentation of the Symposium. The interest in the papers presented by the speakers, and the ensuing discussion, were extremely gratifying. It is hoped that similar programs can be developed in the future to expand further on the topics brought out in this program.

Symposium Committee

W. H. Munse
W. J. Hall
J. E. Stallmeyer

WELCOME

N.M. Newmark, Head, Department of Civil Engineering, University of Illinois

I am happy on behalf of the University of Illinois and the Civil Engineering Department to welcome our distinguished guests and visitors to this symposium. This is an unusual opportunity for us. We feel that we are extremely fortunate to be able to have such a program on our campus. This is possible through the assistance of the Ford Foundation Intramural Program and because of the coincidence with this meeting and the annual assembly of the International Institute of Welding, just recently completed, in New York City. It is the first time that the assembly of the Institute has been held in the United States.

Our speakers include a number of important officials of the International Institute, either as Chairmen of Commissions or as members. They will be introduced in due course. We also have in attendance a number of guests who have been active in the work of the Institute, representatives from a number of companies including Esso Research and Engineering, American Bridge Co., Caterpillar Tractor Co., The Bureau of Public Roads of the United States Government, The National Academy of Science, The National Bureau of Standards and many more.

We hope that we can make your visit to our campus interesting. If you have any questions, or see anything with which we can help you in any way, please do not hesitate to ask help from any of the staff members here or those whom you might meet in your tours on the campus. We have a very active research program in welding and other phases of materials, structures, and mechanics here on campus and in this department. Of our staff whom you have met, Professors Munse, Stallmeyer, and Hall are active in the Institute, and have arranged this program.

It has been suggested that you might be interested in learning something of the Department of Civil Engineering of the University of Illinois. This Department is the largest department of Civil Engineering in the country. We have some 600 undergraduate students, which is somewhat less than some of our sister institutions, particularly those in metropolitan centers, but we have 265 graduate students in Civil Engineering, which is larger almost by a factor of 2 than any other school in the country. Over a period of some 15 years, the number of degrees granted by this institution is in the same proportion as the figures from 1954 to 1959, which are indicative of the general trend. The number of doctorates granted by the Civil Engineering Department of the University of Illinois during this period was 51. The next two schools, Purdue and M.I.T., granted 32 and 25 respectively, Michigan 21, and Columbia 19. During this same period we gave about one-fifth of all the doctorates in Civil Engineering in the country. From 1948 to 1955 we gave one-quarter of the doctorates. Over the whole period we have given something of the order of 10 per cent of all the masters degrees in Civil Engineering, and 2 1/2 per cent of the bachelor degrees.

The Department has 85 full-time staff members and an additional 70 part-time research assistants. Our research program is supported by University funds, in part, and by contract grants in a large part. Contract funds for the last few years run on the order of a million and a quarter dollars per year. Total money spent on research directly, that is not including indirect costs, is somewhat less than this. For this Department of Civil Engineering a direct expenditure of about $918,000 per year was reported as of a year ago. The next institution in the survey that was made last year was California with slightly less and M.I.T. with about $600,000, and so on down the line. During this same period the total contract research in the research effort of the country directly spent on research in civil engineering was of the order of about 10 million dollars. Our institution apparently handled slightly less than 10 per cent of the total.

We have reasonably good facilities. It is difficult to comment on our facilities for our research and instructional program because on the one hand I want to say that they are very good to impress you, and on the other hand I want to say that they are very poor to indicate to the Administration of the University the need for more space, more money, and more facilities. It is difficult to draw this very thin line dividing these two points of view. I think, however, that you will find when you get a chance to look at our activities in the several buildings over which they are scattered, that we are quite crowded. There is a great deal of work going on in a very limited space. Some of it is truly outstanding work. Some of it is work of the sort that simply needs to be done to contribute help in the development of better design and better concepts in civil engineering, but which is not of such long range fundamental importance. I think we have a good balance between the two aspects in civil engineering design, and structural mechanics and materials. Thus we have the proper kind of program.

I won't try to describe much about other activities of the department. There are activities in hydraulics, sanitary engineering, construction, soils, foundations, highways and transportation, as well as structures and structural mechanics. I would like to say, however, that we are in the midst of making plans for long-range developments now with a view toward looking at the kinds of things that need to be done by educational institutions in the future. I expect that some of the types of activities, both in instruction and research, need to be changed. The following may be some of the future problems. It is unquestionably true that the education of engineers is going to have to be based on a much more scientific basis than it has been in the past, with less formal instruction in how to do things and more on why things behave the way they do. We are mindful of this with our plans and their bearing on our instructional program and our research effort.

LABORATORY STUDIES OF WELDED JOINTS

R.P. Newman, British Welding Research Association, Cambridge, England

In the time that is available I cannot, of course, give anything like a comprehensive review of work in this field that is in progress in Europe. For this reason I propose to deal with certain investigations being undertaken in the United Kingdom where experimental work on the fatigue behaviour of welded joints and structures is, to a large extent, located at the laboratories of the British Welding Research Association.

A number of projects are currently in progress, mainly concerned with welded steels. Other materials are not without interest, but there is still an important need to enlarge present knowledge of the steel structure in the context of fatigue. Moreover, it is believed that theory can, at best, make only a limited contribution to the practical business of designing and fabricating a welded structure that is destined for service under fatigue loading. It follows that one is heavily dependent upon experimental testing to evaluate a large number of variables in the form of materials, design detailing, welding procedure and so forth. And it is also necessary to introduce the variables in a fully representative way, preferably by making little concession on specimen size if this would mean departing from the use of actual fabrication practice. Fatigue studies in the field of welding in particular are thus often characterised by the use of large scale test equipment. This, briefly, is the general background to the aims and means in much of the current research, some of which is now reviewed.

TRANSVERSE BUTT JOINTS IN MILD STEEL

The transverse butt joint has, of course, been studied quite extensively. However, we felt a need to consolidate our knowledge by further work because it is apparent - even assuming the complete absence of weld metal defects - that one cannot state a unique fatigue strength value for the joint.

Figure 1 shows the typical mode of failure, with the fatigue crack initiated at one edge of the weld, in this case on the root side. The test programme has established that a wide range of strengths may be exhibited by different groups of joints, all of which fail in this manner, and the inference is that strength is dependent upon the exact form of the joint--or the shape of the so-called "reinforcement."

For axial, repeated tension loading (0/+ cycle) the following strength values were obtained at 2×10^6 cycles:

		lb/sq/in
(1.)	Black, mild steel plate and machined butt welds..........................	35,840
(2.)	As-welded joints made with iron powder, cellulosic and certain rutile electrodes..............................	24,640 to 26,880
(3.)	As-welded joints made with low hydrogen and other rutile electrodes.....................................	17,920 to 22,400
(4.)	As-welded joints made with deep penetration electrodes and by automatic submerged arc welding..........	14,560 to 15,680

By checking the performance of some of the lower strength groups with the weld reinforcement machined off, it was shown that the highest value of strength (35,840 lb/sq/in) could be achieved irrespective of original strength. In addition, the original strength of 14,560 lb/sq/in for the submerged arc welds was improved to 24,690 lb/sq/in by adjusting the procedure to give a more favourable shape of reinforcement. The influence of geometry was thus well established.

In an additional series of tests transverse butt joints made by welding from one side and using a backing bar were examined. These gave a strength (0/+) of 14,560 lb/sq/in at 2×10^6 cycles and it was observed that failure was now initiated at the junction of the weld and backing bar (Figure 2).

The need for control or machining of the weld contour in this type of joint is obviously less important; what is more relevant is that the joint should be recognised as an inherently low strength type. Moreover, failure through weld metal--as happens also in the case of pipe butt welds made on backing rings--should not give rise to any misleading conclusions about the strength of the deposited metal.

Although limitations on butt-weld reinforcement shape have been introduced into industrial inspection systems, it is unlikely that this would ever be the case in the field of civil engineering structures. What is likely is that the data quoted will be used to draw up a specification of butt-weld grades in which high, medium and low permissible stress values will be allowed respectively for machined and non-destructively tested joints, shop joints made in the down hand position, and shop and site joints made in other positions or under conditions of difficult access.

INFLUENCE OF DEFECTS ON FATIGUE STRENGTH

The data established for sound butt welds have also served the purpose of setting up a control basis for studying the influence of defects on the fatigue strength of similar welds.

There can be little doubt that one of the universal problems in welding is when to accept and when to reject welds that contain defects. Arbitrary standards of acceptance do exist but, even so, personal judgement cannot entirely be set aside. Most standards are, for example, very coy about the limits of acceptability when defects occur in combination. But while it is relatively easy to criticise standards, any alternative must be based on a rational system and this can only be developed from knowledge of the effect that defects have on the strength of welded joints. The task, for the research worker, is a formidable one. Obviously there is no single criterion of strength, and many variables other than defects and their permutations will need to be covered.

It is well known that when failure by normal ductile fracture can occur there is a marked insentitivity to weld defects. This was observed for static tests of butt welds in several pre-war investigations. It has recently been confirmed in an elegant piece of research carried out in the United States by Green, Hamad, and McCauley,[1] who have shown the critical limit for porosity, before suffering any reduction of tensile strength, to be of the order of 7 per cent. In static tests of actual joints, the superior yield strength of the deposited metal clearly provides a margin for the ineffectual incidence of defects and as a result, any specification built up from tests of this kind would probably do no more than encourage bad welding. A more severe criterion is fatigue strength; since defects always give rise to stress concentrations, a fatigue test thus measures defect severity in terms of notch effect rather than in terms of reduction of cross-sectional area. Results from fatigue tests could also be used in a fairly wide context, and not only for fatigue-loaded structures, if it was thought desirable to apply a rigorous standard; whether or not this is departing from rationality is an argument that can be postponed for some time!

But with the above ideas in mind the choice of fatigue testing to evaluate defects was made at B.W.R.A. (British Welding Research Association) and the work is now in progress. It was preceded by some exploratory tests on mild steel pipe butt welds[2] subjected to alternating transverse bending. The pipes were 6 5/8-in. outside diameter, with a wall thickness of 3/8-in. Little error was involved in assuming that at maximum fibre positions the stress was uniform direct stress over the thickness of the wall. A series of tests on pipes containing butt welds made on a backing ring produced one mode of failure consistently, i.e., the same as illustrated in Figure 2. Further, the strength for 2×10^6 cycles was relatively low, being approximately $\pm$ 8,400 lb/sq in (a strength reduction factor of about 2.4; similar to that for backed joints in flat plate). It was perhaps not surprising when included defects such as scattered, fine porosity and discrete slag inclusions did not cause any further reduction of strength. Somewhat larger gas cavities ("piping" or "wormholes") and even quite extensive lack of side wall fusion similarly caused no loss of strength, and it was not until the inherent notch at the root was, in effect, made more severe by including lack of penetration that a decisive effect on strength was obtained. Having thus enhanced the suspicion of many engineers that the author was up to no good at all, a more comprehensive programme was planned with at least the knowledge that the environment of the defect was an important consideration.

In this later programme, the specimens being used are transverse butt welds in $\frac{1}{2}$-in. thick mild-steel plate (equivalent to ASTM-A7) and some concentrated work on linear slag inclusions is in hand. First it was necessary to be able to produce inclusions of a given size and distribution in a batch of approximately ten specimens so that the usual S-N determination could be made. Initially this was done by introducing a continuous line of slag across the 4-in. width of the weld and then removing unwanted portions. The slag line was produced by making the second run in the weld of asymmetrical form, leaving a continuous, sharp crevice at one side to act as a slag trap. A covering run was deposited and after reference to a radiograph, portions of the slag line were removed by end milling. Figure 3 shows a triple inclusion specimen at this stage and the ends of the slag line in each ligament are discernible. Welding of the machined pockets had to be done carefully to avoid trapping more slag (also checked by radiography) and the joint was then completed normally. The procedure was slow and expensive but capable of giving good results. It was, however, replaced by an altogether more efficient method in which discrete crevices of a given size and spacing were simply produced by manipulation of the electrode during the second run (Figure 4). Two welding operators have acquired a high degree of skill for doing this and some of their results are illustrated; perhaps the B.W.R.A. is fortunate that it is not a skill widely sought after.

Surprisingly little difficulty with scatter or with a non-uniform mode of failure has been experienced. It has, of course, been necessary to maintain good reinforcement shape so that a positive result is obtained and, on the basis of actual failure from the defects, the following kind of picture is emerging. Taking design stress levels and plotting the limiting curve on an S-N diagram, the intercepts with S-N curves for the defective weld indicate maximum permissible endurances of:

(1) for continuous slag line Figures 5a and 5b: 250,000 cycles of design stress

(2) for triple 3/8-in. inclusions (approx.) - Figure 6: 1×10^6 cycles of design stress

(3) for single 3/8-in. inclusions (approx.) - Figure 7: 2×10^6 cycles of design stress

(4) for triple (Figure 8) and multiple (Figure 9) 1/32- to 1/16-in. inclusions - no restriction on endurance at design stress

Without prejudice to any final interpretation, it would appear that there is a fairly good correlation between size of defect and fatigue strength, a correlation which is only slightly disturbed by the actual number of defects. This is broadly in agreement with observations of the influence of non-metallic inclusions in steel; for long endurance stress levels the largest single inclusion is liable to cause failure, whereas for short life stresses, multiple initiation of cracks--even from small inclusions--may occur. For welding defects there is some prospect therefore that the combined slag/porosity situation will be amenable to treatment. If the same pattern of behaviour is to continue, porosity will be seen to be more important at high stress levels and perhaps relatively innocuous at low stress levels in the presence of a single, but larger, slag inclusion. However, this remains for the future, and in the world of fatigue it is wise to take nothing for granted. In the meantime, we might reflect on the finding that a continuous slag line is little more damaging to fatigue strength than a backing bar. One wonders if backing bars appear in any list of unacceptable defects!

THE SURFACE RECLAMATION OF SHAFTS BY WELDING

Shafts reclaimed by welding probably suffer a higher incidence of fatigue failure in service than any other kind of welded component. This has induced many authorities to ban the use of welding reclamation or otherwise to permit it only as a temporary expedient. Investigations of service failures have revealed very often that welding procedure was at fault and that severe metallurgical damage could be traced to the failure to apply the correct pre-heating, post-heating and welding conditions. Even so, some early test work by Wellinger in Germany[(3)] produced a rather startling result in the shape of a 75 per cent reduction of fatigue strength of small diameter mild steel shafts. Another welding bogie seemed to have been found; such a drastic loss of strength could not be attributed this time to residual stresses--certainly not to those remaining after machining; and surely stress concentrations had been entirely removed by machining?

It is now known that stress concentrations are not necessarily removed and that the problem is again linked with the influence of defects. The need to apply appropriate welding procedures remains; but, beyond this, fatigue strength is dependent upon the soundness of the deposited metal. Figures 10 and 11 show failures in laboratory specimens. Isolated, sub-surface pores (Figure 10) do cause a significant loss of strength, but the more dangerous defects are a transverse slag line (Figure 11) or lack of penetration between adjacent runs, particularly when these are associated with circumferential welding and occur as discontinuities lying normal to the direction of applied bending stress. Much depends therefore on the margin between normal working stress and the long endurance fatigue strength of the defective deposit. If a completely homogeneous deposit is obtained then, for mild steel at least, very nearly the full strength of the virgin shaft can be realised.

This aspect of fatigue has been introduced--albeit briefly--in order to show that it is a simple matter to be deceived by defects. How is it possible, after all, to reconcile the comparative immunity of pipe butt joints to any ill effects of welding flaws with the high degree of sensitivity displayed by a reclaimed shaft? Figure 12 suggests an explanation. Work by Homes[4] and by Masi and Erra[5] has shown a relationship between defect severity and the fatigue strength of defective weld metal of the kind represented by the full curve. If one now sets up on the same scale the fatigue strength of actual joints and components, as determined by the stress concentrations associated with joint form, the intercepts with the curve indicate an increasing tolerance of defects as natural strength is reduced. This suggestion, then, is no more profound than the idea of the weakest link in a chain.

FILLET WELDED CONNECTIONS

Any means for securing improvement of the typical fatigue strength values for fillet welded connections should be a matter of general interest to designers, provided the means are less expensive than the traditional alternative of scrapping fillet welds and using an inherently higher strength joint. Recently, some tests on cover plate specimens with longitudinal fillets (Figure 13) have been carried out at B.W.R.A. These have indicated that maximum strength of the "as-welded" detail depends on the ratio of areas of cover plate to main plate and on the length of the fillets. With equal areas the failure of the cover plate--as in Figure 13--is most likely. Strength values for 2×10^6 cycles of repeated tension loading are approximately 7,800 and 10,000 lb/sq in for cover plate and main plate respectively, and some increase in both appears possible by lengthening the welds. However, an increase of a completely different order can be achieved by the use of methods for inducing a favourable state of residual stress in the zones where fracture is normally initiated. Two methods are available: spot heating and local compression treatment. For steels both appear to give the same results.

In the first stages of this investigation a detail with non-load-carrying fillet welds was used; the specimen consisted simply of a flat plate with longitudinal gusset plate attached each side by fillet welds. Figure 14 illustrates the stress systems induced in a specimen of this type by local compression (a) and by spot heating (b) and (c). In cases (a) and (b) the treatments result in the desired situation of residual compression stress in the area of the notch, i.e., at the ends of the gusset plate. High endurance fatigue performance over a wide range of mean stress values, or ratios of f_{min} to f_{max}, is greatly improved by this state of residual compression (Figure 15--line marked "Pressed"). Situation (c), with the heated spots in the wrong plate producing tensile residual stress in the zone of the notch but with approximately the same thermal treatment, was investigated merely to show that improvement of strength was associated with residual stress and not with any metallurgical modification arising from heating. The tests confirmed this by indicating the same fatigue strength as for the original untreated specimens.

The mechanical method, requiring heavy compression between dies, is obviously similar to the methods that have been used for a number of years by Professor G. Welter at Ecole Polytechnic, Montreal, in his work on spot welds for the American Welding Research Council. The thermal technique of heat-spotting was first applied in the context of fatigue behaviour by Professor O. Puchner in Czechoslavakia.[6] For steel they may well prove to be complementary techniques, but for welded aluminium alloys the mechanical treatment (Figure 16) appears to be more suitable and again produces significant increases of what can otherwise be rather poor fatigue strengths in these materials. The more recent work on the load-carrying fillet welds in mild steel has continued this trend, with increases of 100 per cent and 200 per cent in the strengths obtained at 2×10^6 cycles for repeated tension and alternating loading, respectively. One of the spot-heated specimens is shown in Figure 17, illustrating the reason for the cut-away shape of the cover plate; this is to enable the heat spot to be located correctly with respect to the ends of the longitudinal fillet welds from which failure of the main plate would be initiated.

Three important aspects have been revealed or confirmed by this work. First, the two methods employed can only be used successfully to deal with a localised notch, such as a weld end. A cover plate specimen with transverse fillets would not be amenable to treatment. Second, since the effect upon fatigue strength is produced by residual stress, it can be entirely wiped out when applied stresses reach yield value; the effect is thus seen to operate most favourably when conditions of nominally elastic stressing and long life, in cycles of loading, are involved. Third, reverting to Figure 15, the notable increase of strength for stress relieved specimens when the applied loading is predominantly compressive, suggests that the introduction of a compressive residual stress system enables some of this reserve of strength to be successfully transposed to the tension loading area. This source of potential strength is not apparent in the testing of "as-welded" specimens, presumably because the existence of tensile residual stress masks the intrinsic behaviour of the detail under compressive applied load.

HIGH TENSILE STEELS

One of the most intractable problems in the field of welded steel structures is the failure to obtain any useful reward in fatigue properties simply by increasing the tensile strength of the parent material. It is true that in virtue of higher yield values some application is possible beyond the specification limits for mild steel when high mean stress or short cyclic life conditions are encountered. Otherwise many research programmes have come to the same result, namely parity of fatigue strength between welded joints in mild- and high-tensile steels. This result has recently been reinforced by check tests of nine steels carried out at B.W.R.A. There is a specialised interest here emanating from the pressure vessel industry, where it may be shown that if the load-strain curve is non-linear under individual load cycles, one will obviously be concerned with the criterion of strain range. On this basis, yield properties could be important in relation to fatigue life, since with given scantlings and pressure the range of strain induced--for example at an opening--will be dependent on yield stress. A separate investigation of this aspect is in progress, but the more general problem seems less likely to be resolved, at least in terms of the contribution of the basic properties of parent material.

We have thus recently been diverted into the purely engineering approach of the influence of design and construction in securing fatigue strengths of such a magnitude that a high-tensile steel must perforce be used to satisfy specification limits. Lest this sounds like the introduction to some spectacular achievement, however, let it be clear from the outset that only modest increases of strength are, in fact, being sought in this way. The problem is related to plate girder construction and we are testing model girders (Figure 18) in what we call the "Illinois" machine, this being a copy of the one designed and first put into use at the University of Illinois by Professor Wilson. In keeping with local tradition, the machine is working well with numerous fatigue cracks in it.

In these girders the tension flange is entirely clear of stiffener attachments. This at once leads to improved performance, but we have noted that the use of truly continuous, automatic welding gives the best result. Referring to the stresses on the plane of the web-to-flange weld (i.e., the inner surface of the tension flange) developed under unidirectional, pure bending, the values obtained for 2×10^6 cycles are 20,160 and 26,880 lb/sq in respectively for manually welded and automatically welded girders, these strengths being achieved for a carbon-manganese steel (BS.968) with a yield stress of 51,500 lb/sq in. The 30 per cent increase of strength for the automatically welded specimens is undoubtedly a reflection of the influence of removing "stop-start" points from the weld seam; if so, this feature--which seems capable of practical utilisation--also leads to the possibility of employing the BS.968 steel to advantage over mild steel with its limiting <u>static</u> working stress of approximately 23,000 lb/sq in.

Clearly the important step to achieve is the maintenance of this relative position when the girders also contain web stiffeners. We are following with great interest and admiration the work that is being done in this area at the University of Illinois. It has already provided us with much needed information and our plans for the future have been laid accordingly. They include an examination of the possibility of using the residual-stress techniques already referred to as a means of maintaining high fatigue strength in the stiffener girder, and I am pleased to say that there is industrial interest in this possibility in the United Kingdom. These techniques would, of course, be used to diminish the damaging effect of stiffener attachment welds, both on the tension flange and on the web. They have already been used with good effect in the laboratory to serve just this purpose in connection with the heavily stressed, half-depth stiffeners under the loading points on the girders.

This completes my brief survey of laboratory studies but before closing this talk I must add a further comment. To anyone who is engaged upon the problem of fatigue in welded structures, the Talbot Laboratory of the University of Illinois is a place with which he must necessarily become familiar. I have had the good fortune to enjoy close and friendly relations with the staff here for many years, and it is indeed a pleasure to return to Illinois and a privilege to address this meeting.

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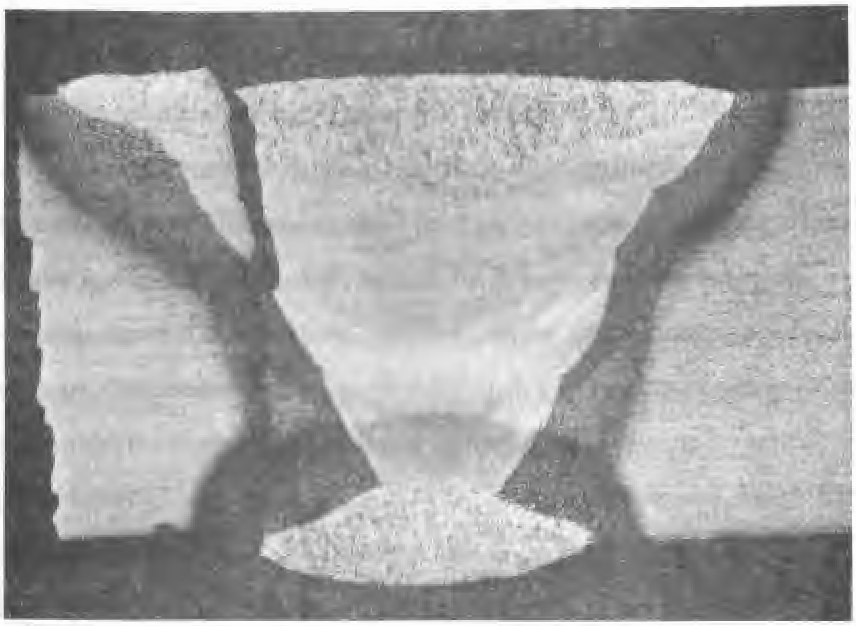

Figure 1   Typical fatigue failure in transverse butt weld

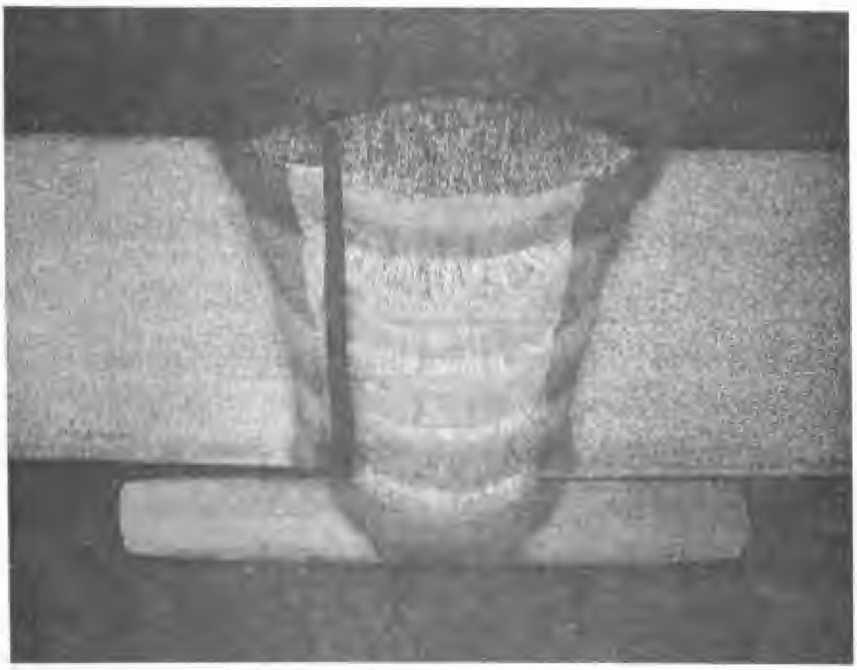

Figure 2   Fatigue failure of transverse butt weld with backing bar

Figure 3 Fatigue specimen machined to leave three slag inclusions in final weld

Figure 4 Fatigue specimen with crevices produced by electrode manipulation

Figure 5a Continuous slag-line (x-ray)

Figure 5b Continuous slag-line (fracture surface)

Figure 6 Triple 3/8-in. slag inclusions (x-ray)

Figure 7 Single 3/8-in. slag inclusion (fracture surface)

Figure 8 Triple 1/32- to 1/16-in. slag inclusion (fracture surface)

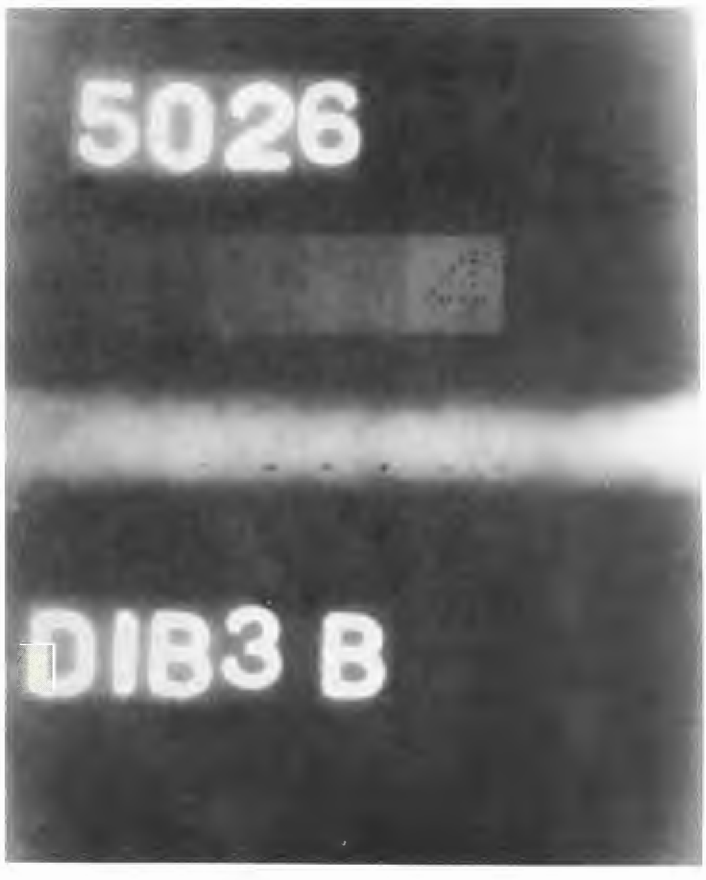

Figure 9 Multiple 1/32- to 1/16-in. inclusions (x-ray)

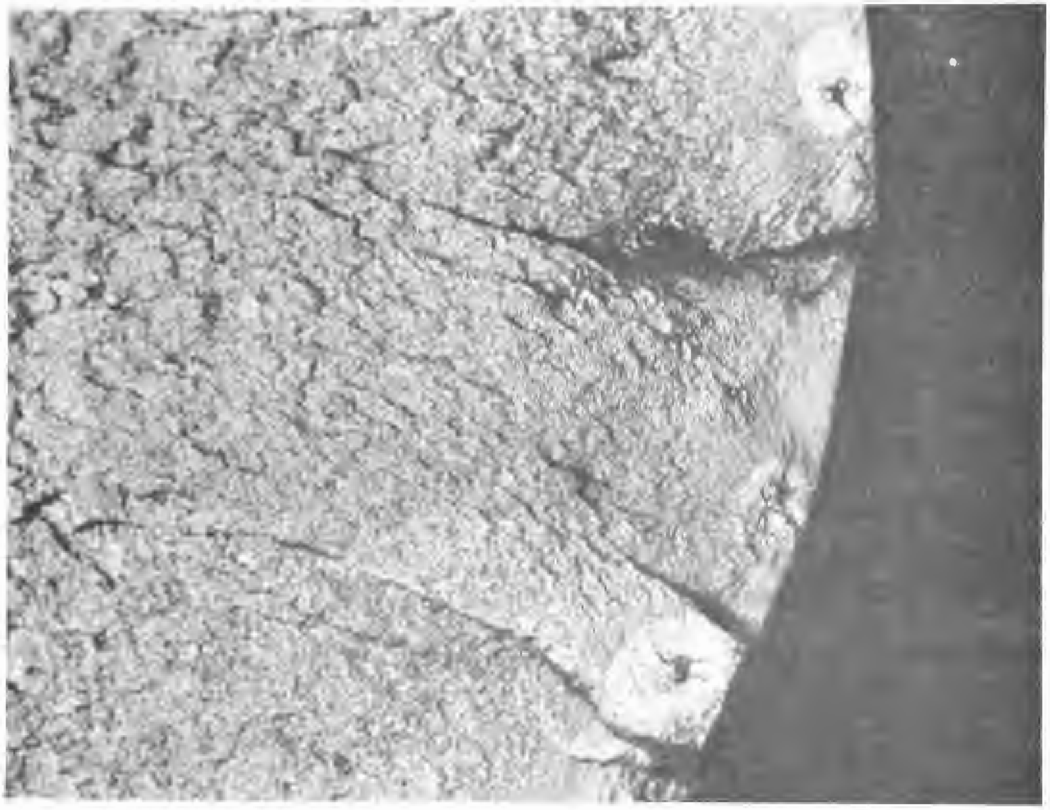

Figure 10   Sub-surface pores in reclaimed shaft (x 4)

Figure 11   Slag line in reclaimed shaft (x 6)

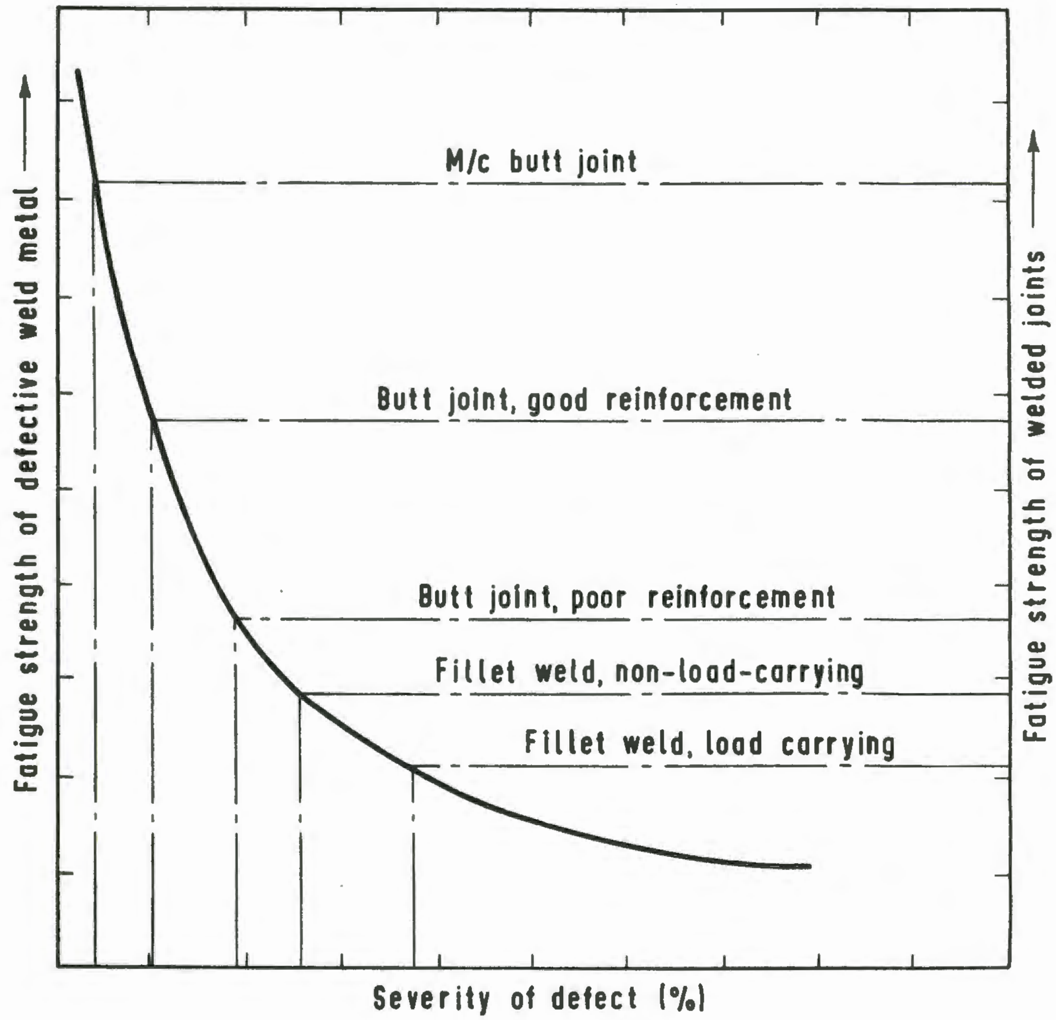

Figure 12 Qualitative indication of critical defect limits for welded joints

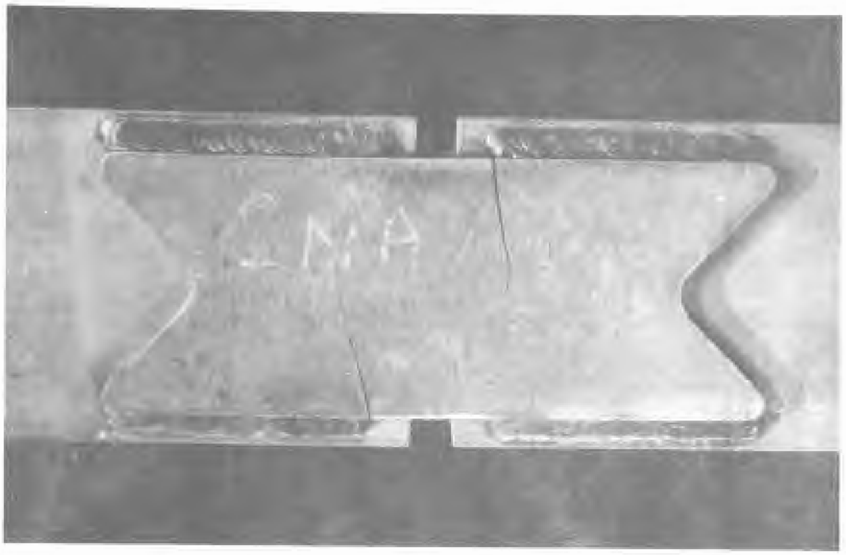

Figure 13 Fillet welded cover strap specimen

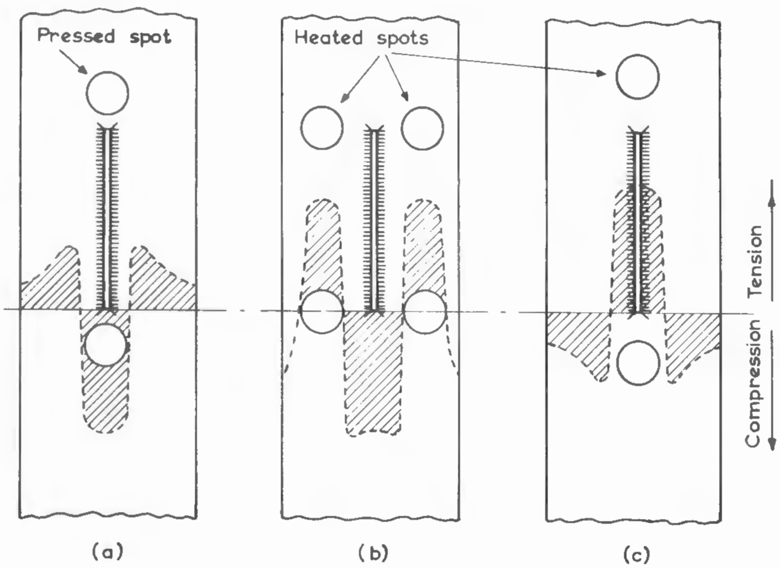

Figure 14 Qualitative distributions of residual longitudinal stress resulting from post weld treatments

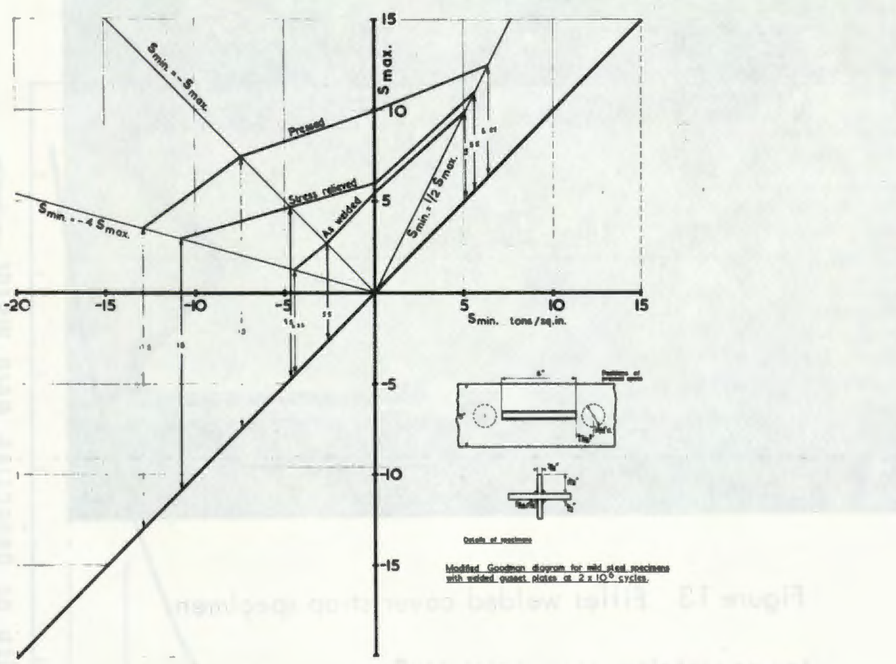

Figure 15 Goodman diagram for mild steel specimens with welded gusset plates (2×10^6 cycles)

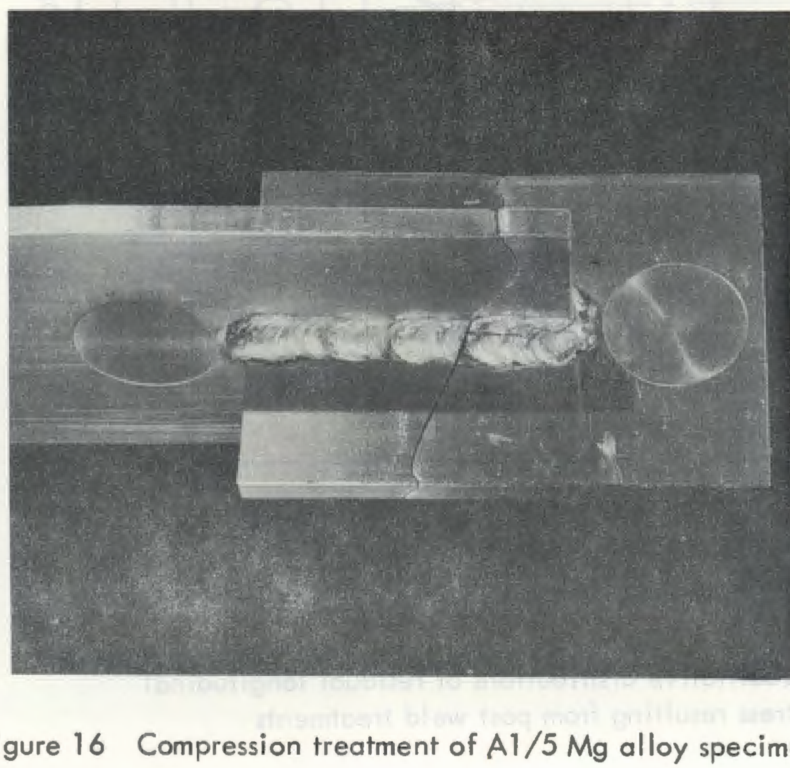

Figure 16 Compression treatment of A1/5 Mg alloy specimen

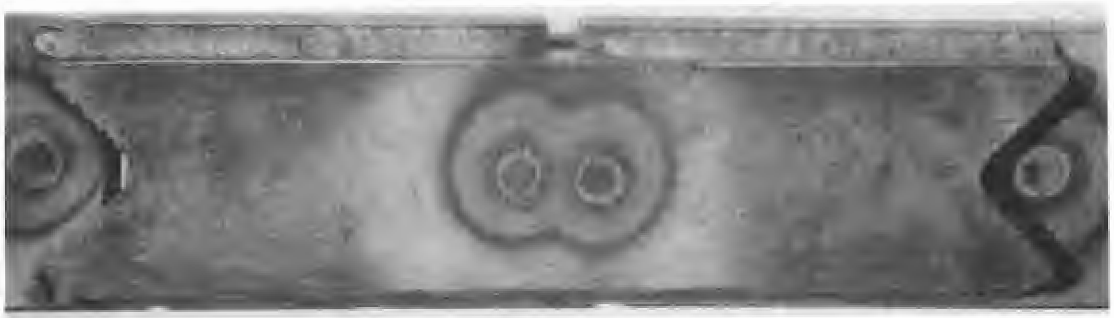

Figure 17 Spot-heated mild steel specimen with cover plates

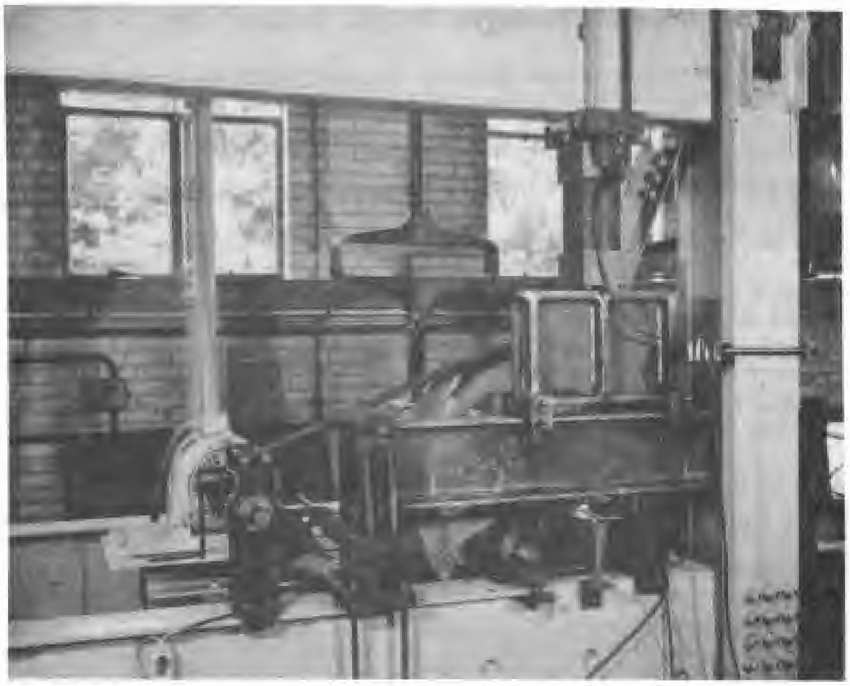

Figure 18 Fatigue test of model plate girder

FATIGUE AND WELDING

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FATIGUE FAILURES OCCURRING IN SERVICE IN WELDED STRUCTURES

As a matter of preliminary quest, let us consider whether welded structures give rise in service to fatigue failures. If such failures were unknown in current practice, the very subject of the fatigue strength of welded structures would be of mere speculative interest and not one to which the engineer should pay much attention.

For some twenty years, there has been much talk about service accidents due to brittle fracture in welded structures, while few, if any, accidents have been attributed to fatigue failures. Would this mean that the frequency of brittle fracture greatly exceeds that of fatigue failures?

Actually, the reverse situation seems to be true; in all likelihood, many more fatigue failures than brittle fractures occur in service. This is easy to grasp, as fatigue failures can only happen in service, while the necessary conditions for brittle failure can be met also in the course of manufacture. Thus, appropriate correcting measures become obvious in due time, which cannot be the case for fatigue failures.

One of the reasons why so much discussion is devoted to brittle failures is that they are generally very spectacular, too often even catastrophic, while the characteristic process of progressive cracking of fatigue failures generally develops with such slowness that incipient cracks may be detected before complete failure. Thus, it is often possible to take in due time the measures necessary to prevent complete failure.

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*This is a brief survey of the work done by Commission XIII--Fatigue Testing--of the International Institute of Welding on the problems related both to fatigue and to welding.

For example, in the case described in 1955[1] a cargo vessel had its rear structure weakened to the point that innumerable fatigue cracks appeared due to an error of design. Nevertheless, the ship could be safely taken ashore and repaired to the full satisfaction of the owner.

Another reason why fatigue failures occurring in service are not often mentioned is that the implied causes are usually more complex than those of a brittle fracture. Moreover, they involve a greater number of responsibilities, with the result that an agreement regarding the release of the information is usually much more difficult to reach between all interested in the case of a fatigue failure than in the case of a brittle fracture. In particular, many constructors fear that discredit will be thrown upon their production if by release of such information it would become known that their structures suffer from fatigue failures in service.

Herein lies a real difficulty for Commission XIII--Fatigue Testing--of the International Institute of Welding. The Commission has been able to get information only after action of several years. After various trials, it now has adopted the following policy.

By circular letter SST-44-54 from the Scientific and Technical Secretariat of the Institute, accompanied by an inquiry form,[2] systematic inquiries regarding fatigue failures in service involving welding or related processes have been undertaken since 1954. This inquiry is circulated under strict anonymity which the members of the Commission agree to maintain. Each year, during the Institute Assembly, the Commission discusses the cases submitted. After editing, publication of these reports is recommended to the Governing Council of the Institute under a number given by the Institute and without any mention of origin or author. The Scientific and Technical Secretariat of the Institute, after a last check of the French and English versions of these reports, sends them to the members Societies of the Institute for publication in their various national welding journals.

This policy apparently provides, in a satisfactory way, the care and discretion desired on the part of the constructor, and which is nonetheless also the desire of those who wish to present a case and to see their efforts published without delay rather than simply being placed in the records of the Commission awaiting future publication in a group of collected papers. To date the Commission has been able to submit for publication twenty-two documents, of which fifteen were submitted in 1958 and 1959.

Considering the initial difficulties, this number of publications represents a real success, although modest, when one considers the number of actual cases of fatigue failures in welded structures.

FACTORS OF FATIGUE STRENGTH IN WELDED STRUCTURE

This collection of twenty-two documents, many of which concern complex structures, is evidently too limited to permit an easy evaluation of the various factors controlling the fatigue strength of welded structures. To look for these factors, let us first consider simpler structures, such as elementary assemblies: cruciform joints, butt joints, etc.

With the present knowledge of welding techniques, such joints in usual structural steels would, in general, fail away from the weld under static loading (Figures 1 to 3). On the contrary, when tested in a hydraulic pulsating machine, for instance as per the standard conditions established by the International Institute of Welding, i.e., in fluctuating tension with the lower cycle limit fixed at 2 kg/mm^2, the failure location is at the edge of or within the weld.

For instance, for a butt joint, cracking always starts at the toe of the weld. If this weld is double-vee shaped, cracking will develop transversely in the parent metal. If the joint is single-vee shaped, the same occurs provided the initiation of the cracking takes place on the face side of the joint. If it initiates on the root side, the crack remains transverse but is found to be in the weld itself.

A cruciform joint also gives way to cracking along the welding line in the parent metal. In particular, this is the manner in which plates with welded attachments fail.

On the other hand, when the load is applied to the attachments, a different process of fracture may occur. In this case cracking originates under both welds located on one side of the continuous element and then progresses across these welds, cutting them nearly through their smallest width.

Comparing their lives with that of the parent metal for 2×10^6 cycles, the fatigue strength is nearly halved for a butt joint, and reduced much more for the cruciform junctions.

GEOMETRICAL NOTCH EFFECT

Seeking an explanation of such observed facts, it seems at once to be a geometrical notch effect resulting from the shape of the joint. This explanation suits well the fact that intricate designs, such as the cruciform joint, for instance, provide a greater decrease in strength than the more continuous designs, such as the butt joint. It is also compatible with the knowledge that the so-called "comma" cracks, which might appear along the weld in quench-sensitive steels, are extremely detrimental to the fatigue strength of butt joints (Figure 4).

METALLURGICAL NOTCH EFFECT

However, for a butt joint without defects, the geometrical notch effect does not seem sufficient to account for the large decrease in fatigue strength from that of the parent metal. It must then be admitted that something else comes into the picture, which could be called a metallurgical notch effect. This weakening factor is linked with the very presence of the weld: first on account of its own structure; second because structural alterations of the parent metal are brought about by the welding and probably also because a residual stress field is induced by this operation. That a metallurgical notch effect exists independently from the geometrical notch effect is confirmed by finding that machining a butt weld to restore the smooth external shape is not sufficient, in general, to remove the fatigue crack from the weld. Actually, this fracture is sometimes located in the weld itself, and not between the weld and the parent metal; this is, for instance, what is often observed on mechanical parts built-up by welding. But, as shown by a thorough metallurgical investigation,[(4)] the origin of the cracking is then, if not in a defect of the weld (blowhole, etc...), at least in a zone where the deposited metal has been annealed by an adjacent or superimposed welding run (Figure 5).

The residual stress influence is more difficult to study. These stresses, the measurement of which is always difficult, probably vary during the fatigue process itself and, anyway, will tend to vanish when the failure occurs. Thus, it is usually in an indirect way, in fact through the welding sequence, that these stresses are described when considering their influence on fatigue strength. Though this problem is far from being clearly understood, it is presently out of the question to assume that the welding sequence acts, sometimes very distinctly, upon the way welded structures withstand fatigue.[(5)]

RIGIDITY EFFECT

Leaving the elementary joints, let us now discuss the welded structures themselves. Obviously, the notion of metallurgical notch effect is important, inherent as it is to the welding itself.

For the geometrical notch effect, this notion also holds; but, whereas for the elementary joints it was simply linked with a more or less continuous shape, another aspect has to be accounted for here, namely the efficiency of the junction between the two parts, each of which has its own function in the structure. These junctions are far more rigid in a welded structure than in any structure where the joints would be made by some other process, such as bolting, riveting, etc. The increase of rigidity in the welded structures produce two important consequences, representing together what can be called the rigidity effect.

First, in the junction zones of the welded structures, there often appears very high stress. In an example quoted by Weck,[6] a welded junction at the end of a beam is, for this beam, a true clamping and not a simple support. In this case, the moments are integrally transmitted from this beam to the part of the structure to which it is attached, thus producing an overload on this part (Figure 6).

Second, for the same distribution of mass, a welded structure will have a higher frequency of vibration and a somewhat lower damping capacity than, for example, a riveted structure.[7] This often tends to facilitate resonant vibration of the structure under periodic forces.

SOME PRACTICAL EXAMPLES OF FATIGUE FAILURES ON WELDED STRUCTURES

The main factors affecting the fatigue strength of welded structures having thus been recognized, it is now possible to return to the reports of fatigue failures already gathered by Commission XIII of the International Institute of Welding, with a view to finding among them some typical examples of the action of these various factors.

EXAMPLES OF METALLURGICAL NOTCH EFFECTS

Let us consider a mechanical part behaving correctly in service. As noted above, it is not rare that when such a part is surfaced by welding, and even if it is brought back to exactly the same design, it would fail by fatigue. For this typical case of metallurgical notch effect many examples have been reported by various authors. For its part, Commission XIII has described in document No. 20-59[8] the fracture of a drive shaft of a rolling mill reduction gear and under document No. 12-59[8] the fracture of an engine crankshaft. In this last instance (Figure 7), the cracks originated in a fillet (thus at a geometrical notch effect) but in absence of any building-up there is no fatigue failure.

As the building-up ends in the fillet, this shows that the fracture is actually due to the superposition of both metallurgical and geometrical notch effects.

Let us mention in passing that these unpleasant findings on parts built up by welding have lead Commission XIII to survey the research in progress in various countries to select building-up techniques leading to the smallest reductions in fatigue strength.

Actually there is no need for the build-up to extend to areas as large as in the cases already quoted to provoke a premature fatigue failure. On the contrary, very small welds are sometimes more dangerous than very large built-up welds; the drastic cooling produces a quenching of the metal which may cause cracks to develop. A very curious and relevant case has been recently submitted to Commission XIII. It refers to a 0.35 per cent carbon steel shaft which has been struck by a spatter of fused metal while stored in a welding workshop; thus a local quench occurred and cracks developed out of which a fatigue fracture has progressed.

On the same subject, it is known that nothing is more detrimental to the fatigue strength of a welded structure than the striking of an arc at some place outside of the joints themselves. The signs visible on the reactor element of Figure 8 could be rightly reproduced on all welded structures that are submitted to cyclic stresses.

This detrimental effect of small welds has been often totally ignored by manufacturers or users, and the survey of Commission XIII has brought to light several clear examples of this disregard; in one case (document No. 37-59, to appear soon) the weld, not larger than a drop of metal, was nevertheless sufficient to provoke the failure of a medium carbon steel lorry frame longitudinal member; whereas, for some others (documents No. D (9) -1959[8] and 41-59, in course of publication), the fillet welds joining various attachments have caused the same effect for a lorry front axle, a trailer axle and a lorry underframe (Figure 9).

EXAMPLES OF RIGIDITY EFFECTS

Figure 10 shows the axle arm of a washing machine which has torn away from the central part of the flat end of an 18-8 stainless steel cylindrical vessel. Fatigue fractures were initiated at the end of each gusset plate welded to this vessel to reinforce it. In this case, described in document C - July 1956,[9] the metallurgical notch effect was presumably not very marked, due to the particular nature of the structure. But the rigidity effect is clear; at the ends of the gusset plates the surface of the flat end of the vessel is locally submitted to high bending moments to which it cannot offer a sufficient resistance. The situation would be much improved if the gusset plates were to join the periphery of the end plate where this plate is reinforced by the cylindrical wall of the vessel.

Another typical case concerns a compressor piston of a free piston generator (document No. 38-59, in course of publication) whose hub is linked to a rim through eight radial ribs. Each of these ribs is joined to the rim by a fillet weld running around along its end; here again the rib wall has to withstand locally (at the junction of the ribs) bending stresses exceeding its fatigue strength (Figure 11). The main remedy has been to increase the wall thickness in order to enhance the flexural rigidity of the rim.

The other problem of the rigidity effect in welded structures, namely the ease with which it can be set in resonance, is often more difficult to combat. Only rarely is it possible to break the resonance tuning by acting on the exciter cause, as this involves, for instance, arrangements as expensive as the replacement of an engine by another with a different number of cylinders.[10] Commission XIII has thus had to consider systematically the damage of hull plates observed on 650-ton motor ships that none of the attempted repairs could stop except for the replacement of welded joints between floor frames and bottom plating under the engine by riveted junctions with double angle irons (document No. 39-59, in course of publication).

EXAMPLES OF GEOMETRICAL NOTCH EFFECTS

The notch effects of many different types of geometry can be met in welded structures.

One of the most dangerous is no doubt the one resulting from incomplete root penetration. The non-penetrated part of the joint is, in fact, equivalent to a sharp and deep notch, ending in a region where the structure is not always perfectly compact and where substantial residual stresses often prevail. A typical example, submitted to Commission XIII in document Y,[(6),(9)] concerns a hollow mixer arm of cast stainless steel, where holes for core supports had been sealed by plugs butt-welded from the outside and which failed at the periphery of these plugs. Even full penetration of the weld might not be sufficient for satisfactory mechanical continuity. For instance, too drastic a change in thickness might suffice to induce fatigue failure under cyclic stress. Under reference Z, Commission XIII has published a report[(9)] of a fracture of a pipe directly welded on a flange which is bolted on a rigid machine. Merely by insuring a smoother thickness variation further incidents were prevented.

A more dangerous type of discontinuity is one which transmits the force through a twisted path. For structures made up from rolled products (sheets and rolled sections) such as ship hulls, the need for ease of construction[(11)] or water tightness of some members might sometimes induce unfavourable designs in this respect. Commission XIII's survey has brought out two very relevant examples.[(8)] One of them (document No. 16-59) refers to the connections between longitudinal bulkheads and transverse bulkhead stringers in a tanker (Figure 14); the other (document No. 14-59) refers to the intersection of the ballast platform and the web frames in the forward part of a banana ship (Figure 15).

The desirable smoothness of design might be very difficult to achieve for some kinds of assemblies. This seems often to be the case for tubular structures, either for the intersection of two tubes, even of different diameters, or for the prolongation of a tube by a flat iron.

MEANS OF IMPROVING FATIGUE STRENGTH OF WELDED STRUCTURES

The problems linked with the manufacture of welded structures having to withstand fatigue stresses should not be disregarded. But, conversely, it must not be thought that welded structures are systematically unfit for good service under such stresses. On the contrary, many of the answers received by Commission XIII in the course of its inquiry not only state observed failures, but also describe efficient remedies by which recurrences of such failures have been avoided.

IMPROVEMENT BY DESIGN ALTERATION

Among the cases already quoted, the example of the compressor piston of a free piston generator is quite typical. Here a mobile part of extremely complicated design, due to the complex role played by this machine part itself, does not, as it is now designed, give any rise to failure. In another related area, it is also known that judicious design provides a satisfactory endurance to welded diesel motor frames.

Moreover, it might happen that the best design against fatigue could not be obtained with the very first alteration attempted. Developments towards the highest endurance might proceed by steps. Examples of such step-by-step processes have been given by Weck[6] for open gap press frames (Figure 16) and by Commission XIII (document No. 40-59 in course of publication) for welded water tanks of locomotive tenders.

IMPROVEMENTS WITHOUT DESIGN ALTERATION

Of course, it is not always possible to consider important changes in design. It would thus be useful to have other means of improving the endurance of welded structures, i.e., of bringing the fatigue strength of the joints existing in these structures to higher values than the comparatively low ones already quoted. Along which line should this search be oriented? To find out, let us come back to our former analysis of the factors involved in the fatigue strength of welded structures.

Now, assuming that the design does not change, the rigidity effect also remains unaltered. The geometrical notch effect itself can only be slightly lowered after welding and then by such costly means as machining. This, however, is often not acceptable, from the economical point of view, except for the mechanical parts which by nature would have to be machined in the shop. Hence, except by recourse to such special processes of welding likely to improve the shape of the weld surface, the only factor to consider here is the metallurgical notch effect.

Yet this metallurgical notch effect depends, as already indicated, on the structural state on one hand and on the stress state in the welded region on the other hand. But, as far as the structural state is concerned, there is probably not much to gain. For example, a regeneration of the structure by annealing--supposing that it would be feasible, might do more harm than good. As already shown in case of surfacing by welding, the weak points where the cracking originated corresponded to the zones where the deposited metal had been annealed by the subsequent passes. The search must then be focussed essentially on the stress field.

The first idea which comes to the mind is to reduce or eliminate the welding stresses, as their effect is, in general, harmful to endurance. Actually, thermal relaxation of the stresses often leads, on the laboratory scale, to some increase of the welded structures endurance.

This general thermal relaxation of the residual stresses is by no means selective, whereas the conditions necessary for the initiation of the fatigue cracking process are only met locally. It would then be much better to act upon the stresses also in a local area. Since, in general, the weak points are at the same place as the geometrical notch effects, this action could also be oriented towards using the residual stresses to fight the said notch effects, i.e., towards raising favourable systems of stresses, replacing the natural unfavourable ones.

Along this line, Commission X of the International Institute of Welding, which deals with residual stresses and their relaxation, has had its attention drawn during the last few years to the work done in Czechoslovakia by Puchner.[12] He has shown that fatigue fractures originating at the tips of brackets laterally welded to longitudinal members of, say, a bridge structure, can be eliminated with appreciable increase of the fatigue strength. This elimination may be obtained by way of a local heating of the longitudinal member; the zones prone to cracking being thus placed under compression stresses. A number of variations of this process have been developed for different types of connections. Some of these, studied in particular in Great Britain by the British Welding Research Association, make use of a local mechanical compression instead of the local heating called for by Puchner (Figure 18).

The interest raised by these methods has been such that a thorough study of them is presently being contemplated jointly by Commission X and XIII -- Fatigue Testing--of the International Institute of Welding. However, these processes cannot be adapted to the common cases in which the welded joint extends in a direction transverse to the stress, as butt joints, transverse welded attachments, etc... For such cases, on the other hand, hammering with a multiple tool (Figure 19), suggested by Nacher[13] as early as 1954, with a view to putting a thin superficial metal layer in the weld and the neighboring region into compression, provides a regular and substantial increase of the fatigue strength.[14] This increase can reach at least 25 to 30 per cent for butt joints as well as for sheets with welded attachments, providing the lines of welding to be hammered do not show serious irregularities which might cause an overlapping of the metal under the multiple tool, or prevent this tool from reaching some regions of the weld.

FATIGUE TESTS ON WELDED STRUCTURES

Actually, when figures such as those previously quoted are given for parts of welded structures, they nearly always refer to results obtained by fatigue tests of the classical type, i.e., in which the stress cycles are identically repeated during the whole duration of the test. The performance of such tests raised problems which must not be disregarded.[15] One of these, to which Commission XIII has given full attention, is the calibration of the testing machines; upon its solution[16] lies the progress realized in testing machines during the last ten years.

But more difficult than the technological problems is the scientific one regarding the use of the results and the estimate of their significance with regard to actual service conditions.

Qualitatively, there is no doubt that, as for any other part, classical fatigue tests can be trusted to locate the weak point of a welded structure.[17] Thus, when with the help of Puchner local heating, the fatigue failure can be changed from the tips of the bracket (Figure 17) to elsewhere in the welding line, it indicates that such results will also be found in service, with some increase of the fatigue strength as a consequence.

But quantitative estimates look less certain. This can be attributed to various reasons; only two will be described here.

On one hand, fatigue tests on structures are by necessity often performed not in real dimensions, but on reduced scale models. To the classical scale effect, which appears in metals like mild steel or duralumin, and which always leads to an optimistic estimate of the fatigue strength, must be added the fact that it is practically impossible to reproduce the weld in similitude, with its variations in structure and its own residual stress field.

On the other hand, except for such products as diesel engine frames, where the actual stresses can effectively be represented with good accuracy by the classical fatigue test, welded structures such as bridges, cranes, vehicles, etc., are subjected in service to stresses which vary according to much more complex laws. Rules for the computation of cumulative damage corresponding to such laws are unknown at present. However, it has been established that the stress cycles which are below the fatigue limit corresponding to the classical testing conditions have to be taken into account if they are associated with cycles that exceed this same limit. Furthermore, considering the role of the residual stresses in the process of fatigue in welded structures, the mechanical relaxation through plastic yielding which might be due to an instantaneous overload may modify appreciably the behaviour of these structures. This would affect the efficiency of such processes as those of Puchner and Nacher for the improvement of fatigue strength by favourable stress systems.

These considerations show how important it is, when looking for quantitative comparisons of the fatigue strength of various welded structures, to use test methods that are able to reproduce actual stresses more faithfully than the classical tests. Such methods exist; they are the simulated service tests in which the stress cycles vary during the test, instead of remaining always the same, generally with a periodical manner established beforehand (program fatigue testing).

CONCLUSIONS

As presented, rapid and sometimes superficial, this account will have succeeded, I hope, in showing how complex the subject Fatigue and Welding is, and how numerous are the questions linked with this subject and awaiting a solution.

However, only the case of steel has been considered explicitly or implicitly. Not that the same problems do not exist for other alloys, but actually, in the present state of affairs, it is for welded steel structures that they are more eagerly studied.

The examples quoted show that, in its domain, Commission XIII of the International Institute of Welding has already completed considerable work and, making this statement, it is my pleasure to pay tribute to the active collaboration of the various national delegations in this Commission--collaboration which has efficiently helped to reach some of the results now acquired.

But much remains to be done, and facing these tasks, it certainly is a wonderful encouragement for the Commission to see the interest shown by this Welding Symposium in what is the very subject of its work.

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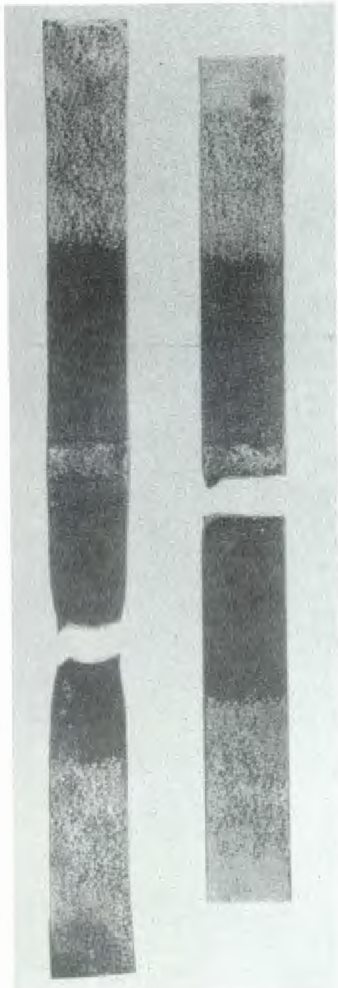


Figure 1 Butt welded joint (x 0.25)

$60\ kg/mm^2$ steel - Nominal thickness: 10 mm
To the left: test piece broken under static loading ($UTS = 62\ kg/mm^2$)
To the right: test piece broken under repeated tensile loading of $2 - 25\ kg/mm^2$

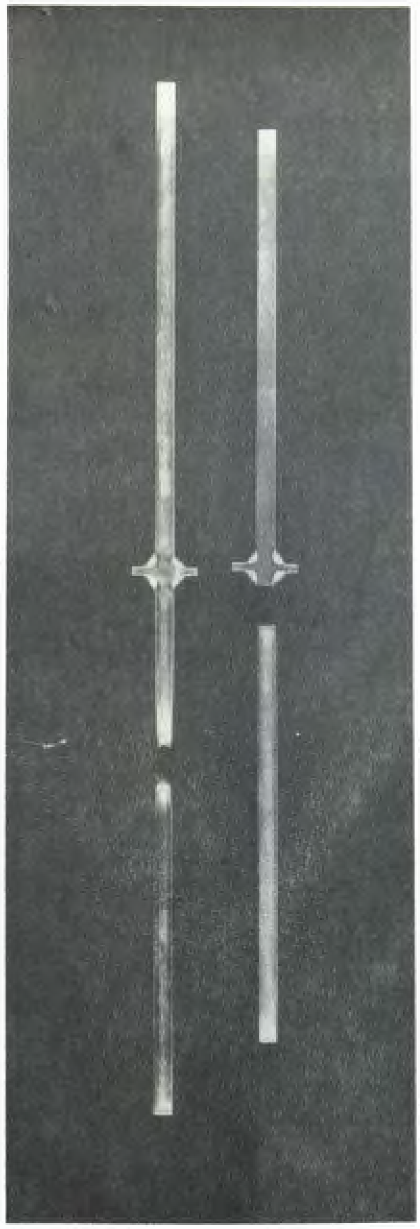


Figure 2 Cruciform welded joint (x 0.25)

60 kg/mm^2 steel

10 mm thick sheet with 5 mm attachments welded on

To the left: test piece broken under static loading (UTS = 55 kg/mm^2)

To the right: test piece broken under repeated tensile loading of 2 - 20 kg/mm^2

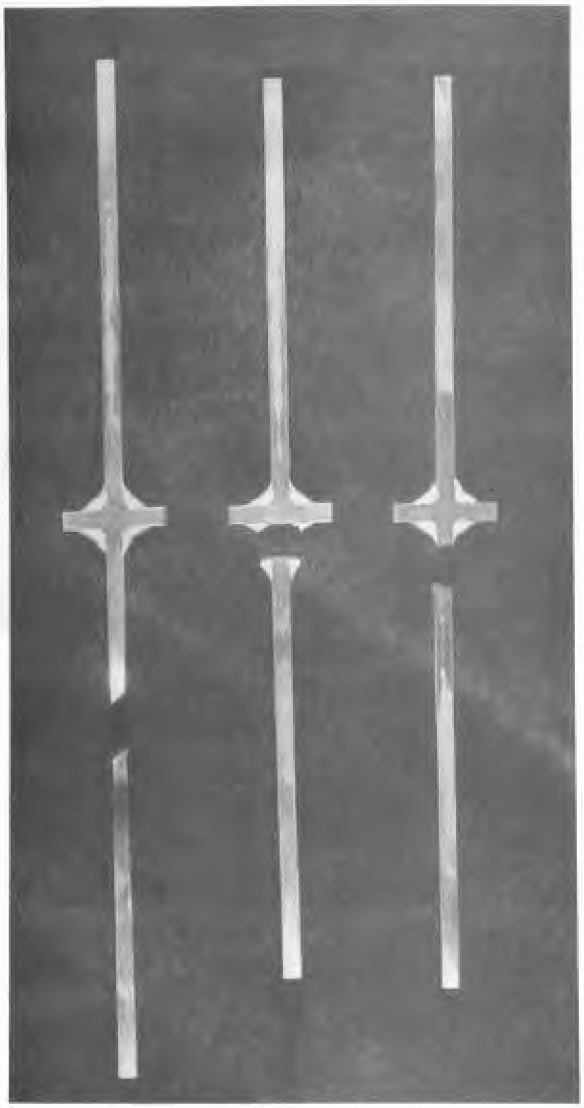


Figure 3 Cruciform welded joint between 10 mm sheets
(x 0.25)

60 kg/mm^2 steel

To the left: test piece broken under static loading applied to the attachments
(UTS = 57 kg/mm^2)

To the right: test pieces broken under repeated tensile loading of 2 - 20
kg/mm^2 applied to the attachments

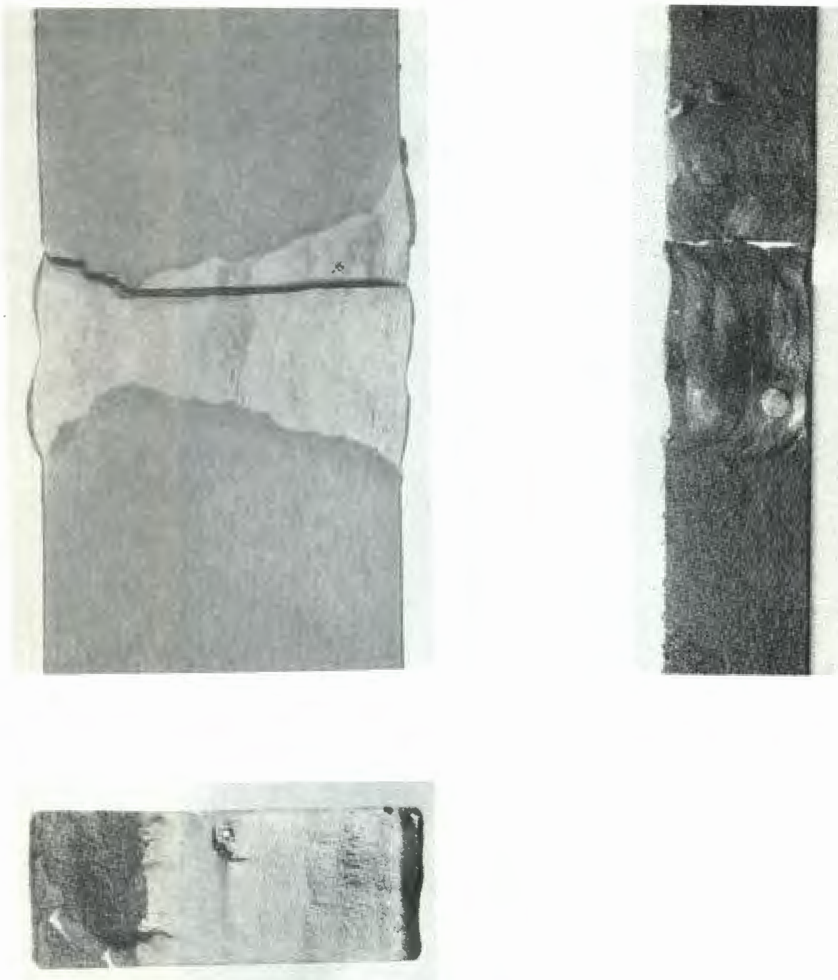


Figure 4 Butt welded joint (x 1)

Nickel-chromium-molybdenum steel (UTS = 70 kg/mm^2)
Nominal thickness: 50 mm
Test-piece broken under repeated tensile loading under 2 - 6 kg/mm^2

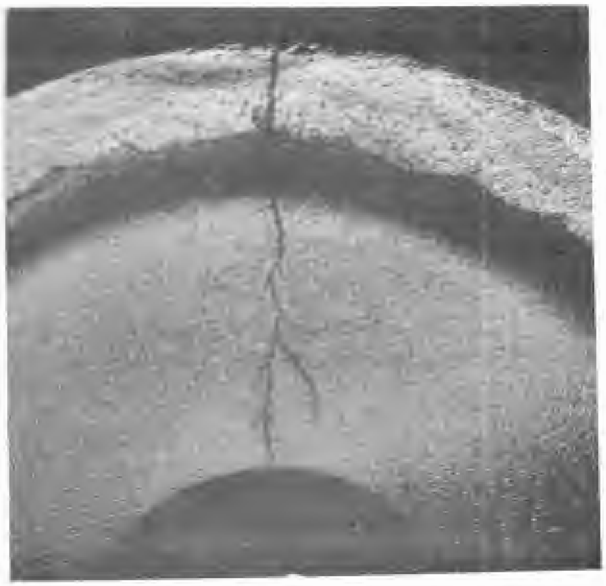


Figure 5 Section of a cylindrical test piece built-up on its periphery and broken under alternating torsion ($\pm$ 16 kg/mm^2)

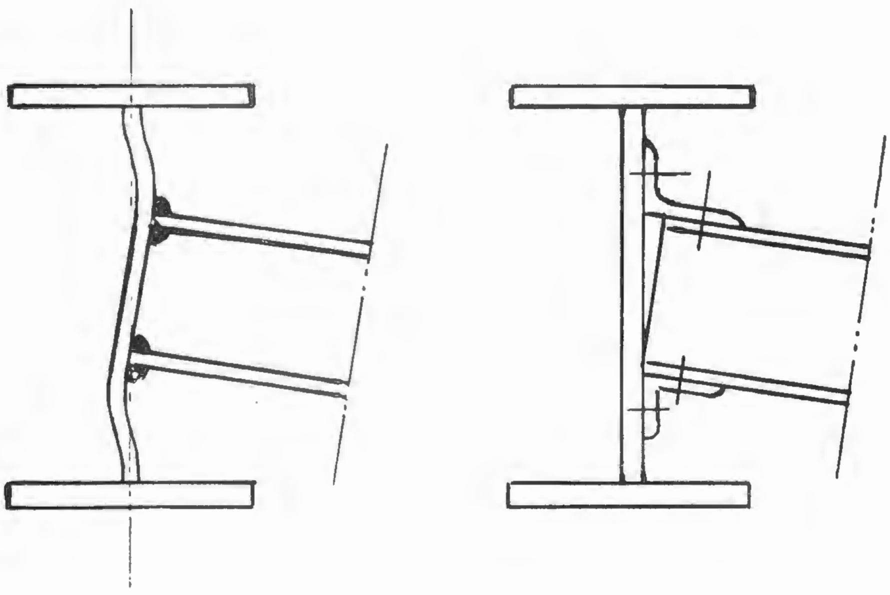


Figure 6 Difference of behaviour between a welded and a riveted junction (after Weck)

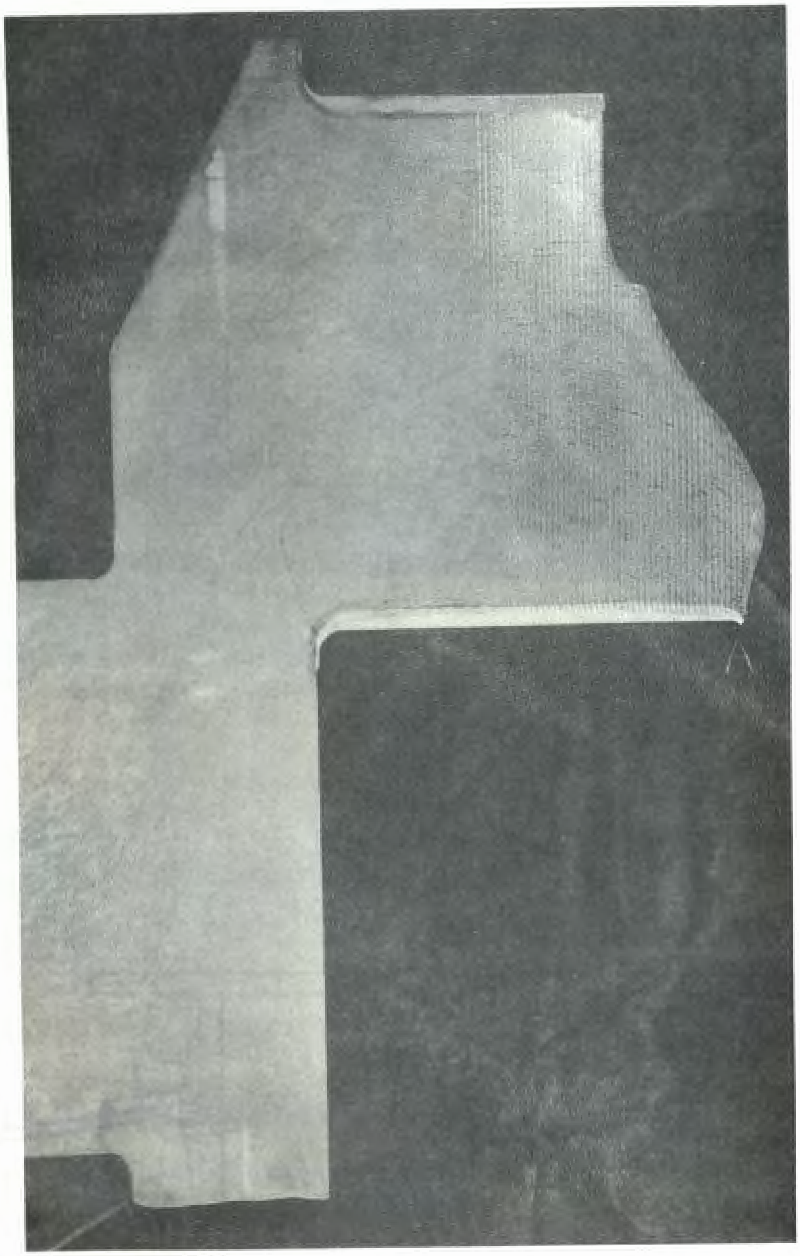


Figure 7 Engine crankshaft, broken between a flange and a crank pin, built up by welding. Section through the plane of symmetry. Etched with iodine.

Figure 8 Shippingport - Central Station Reactor (from materials in Design Engineering, No. 166, January 1960)

Figure 9 Fatigue failure of a lorry frame longitudinal member due to the welding of small attachments

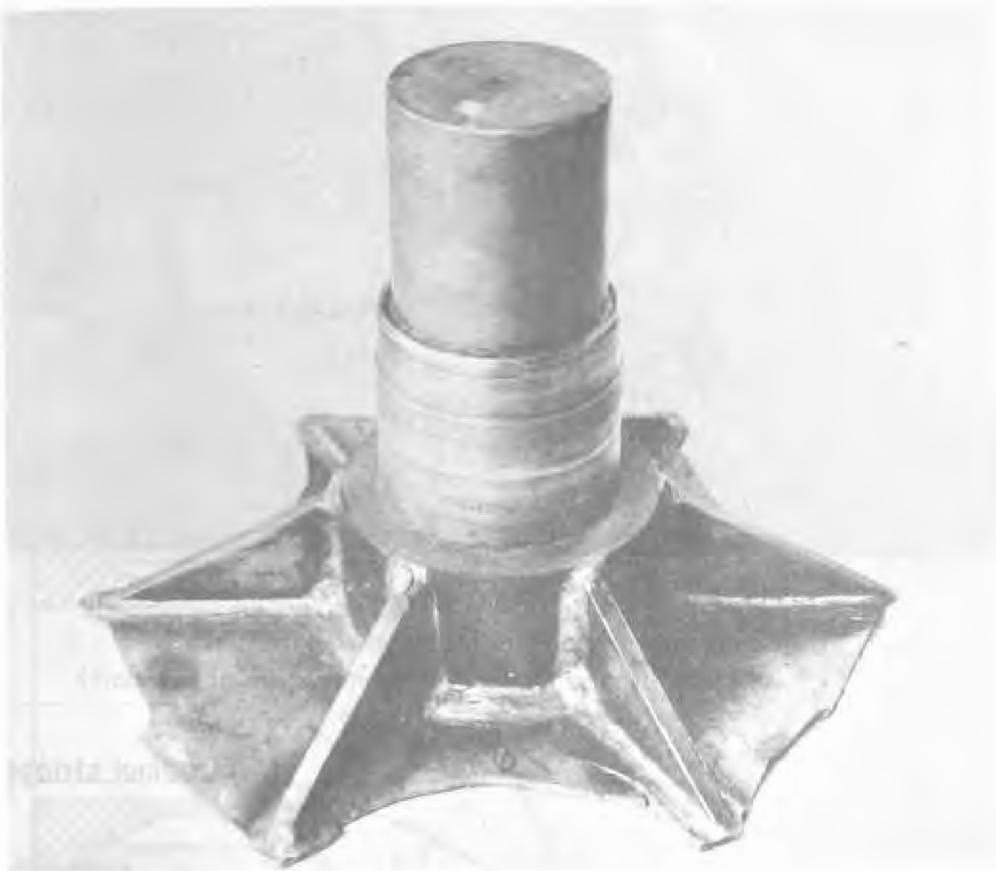


Figure 10 Failure of a washing machine axle

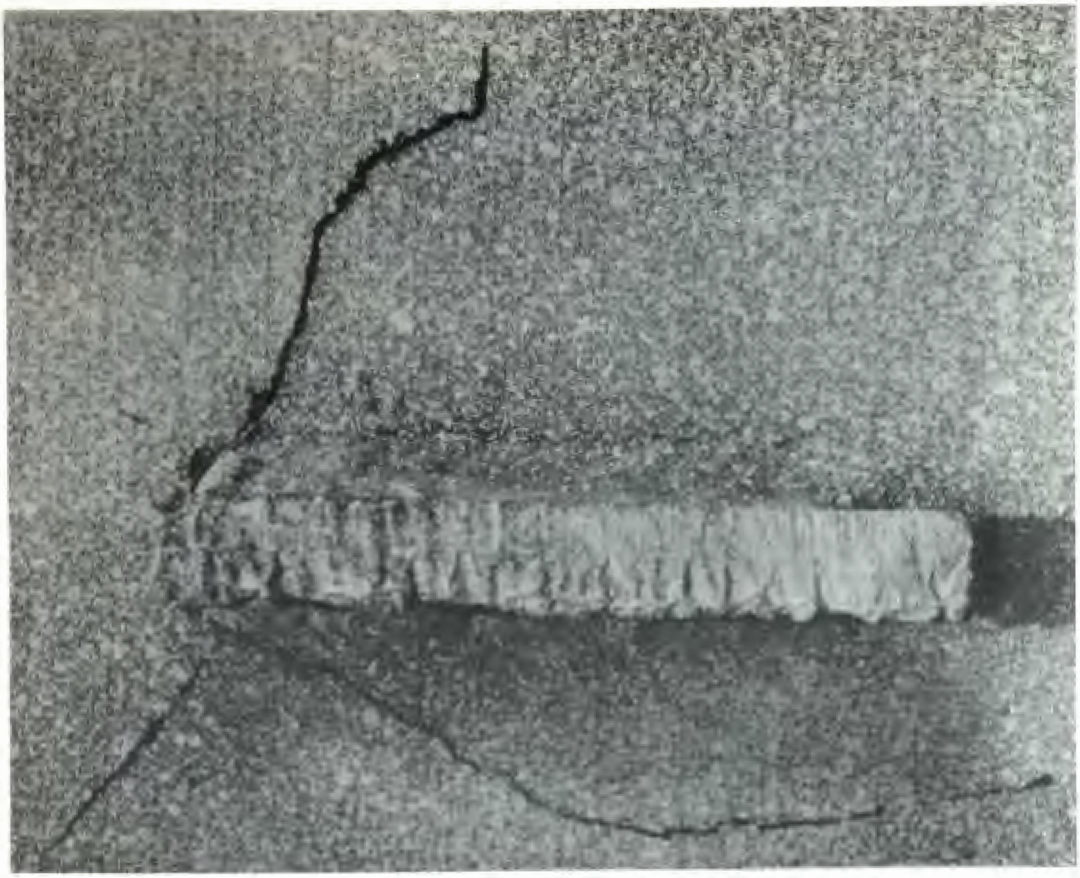


Figure 11 Compressor piston rim fracture at a radical rib junction (flame cut for the examination of the part)

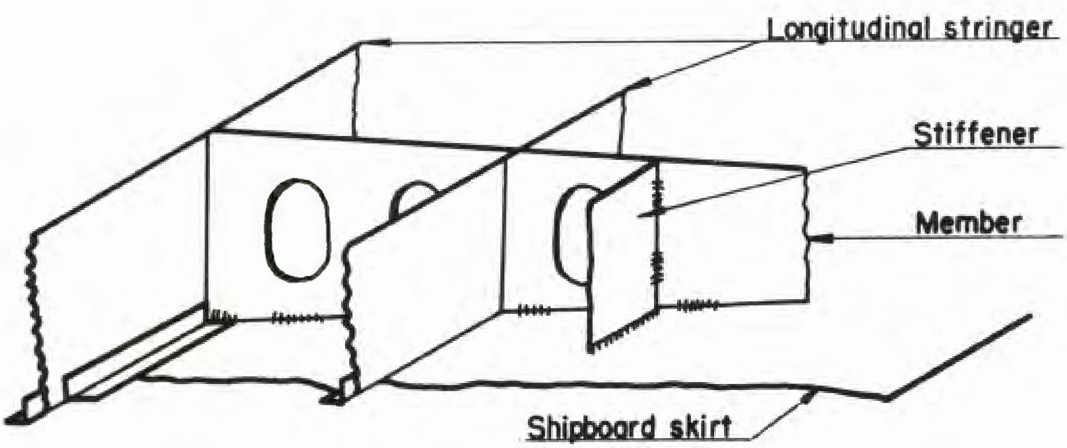


Figure 12 650 tons motorship.
Typical structure below the engine.

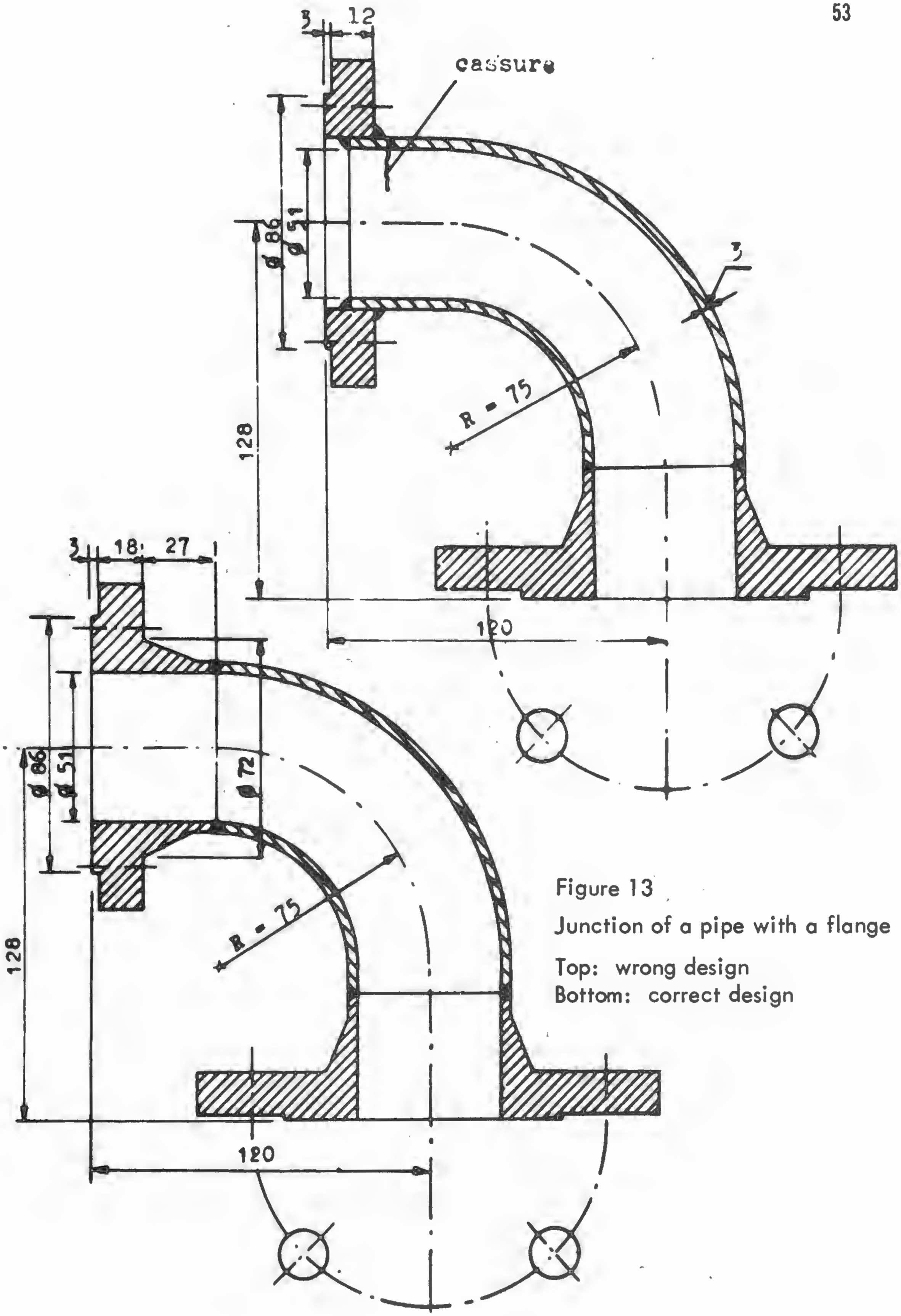


Figure 13

Junction of a pipe with a flange

Top: wrong design
Bottom: correct design

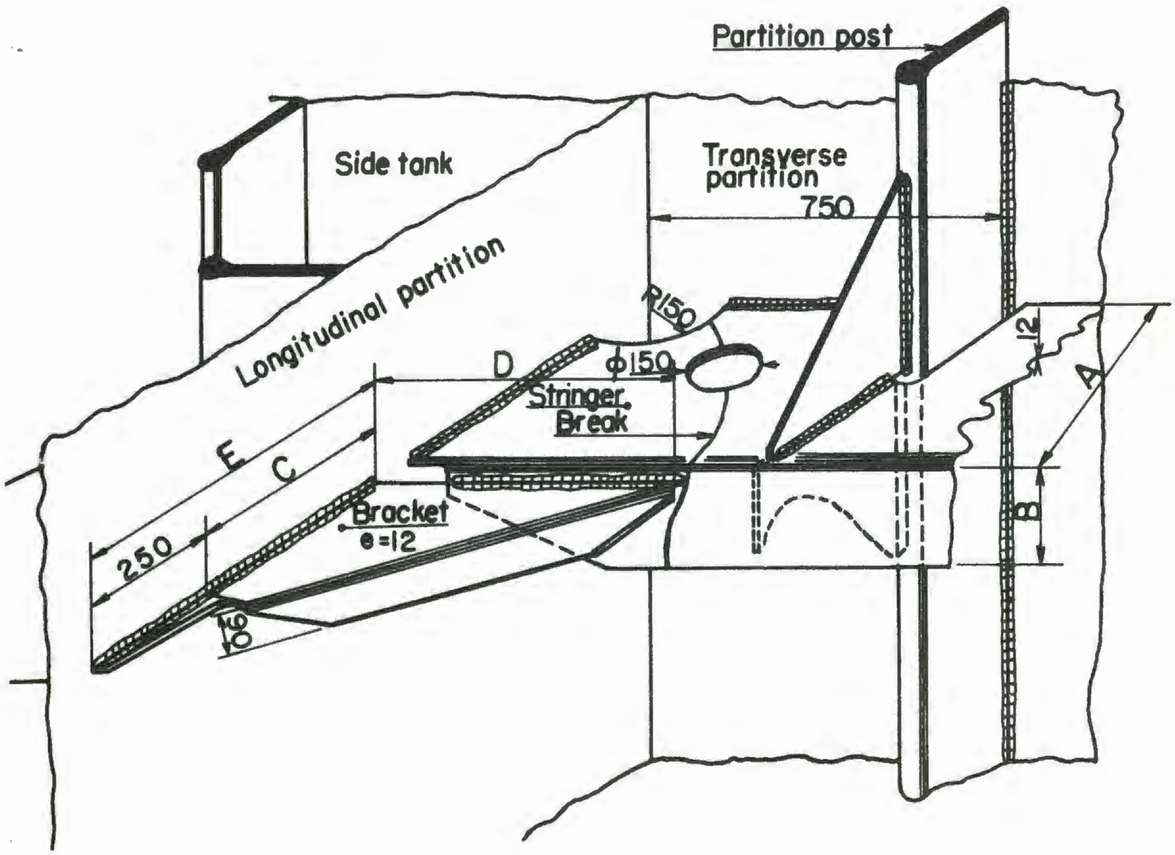


Figure 14 Tanker. Detail of a tank corner.
Note: The bracket is not attached in the plane of the web of the stringer, but on the flanged edge of this stringer.

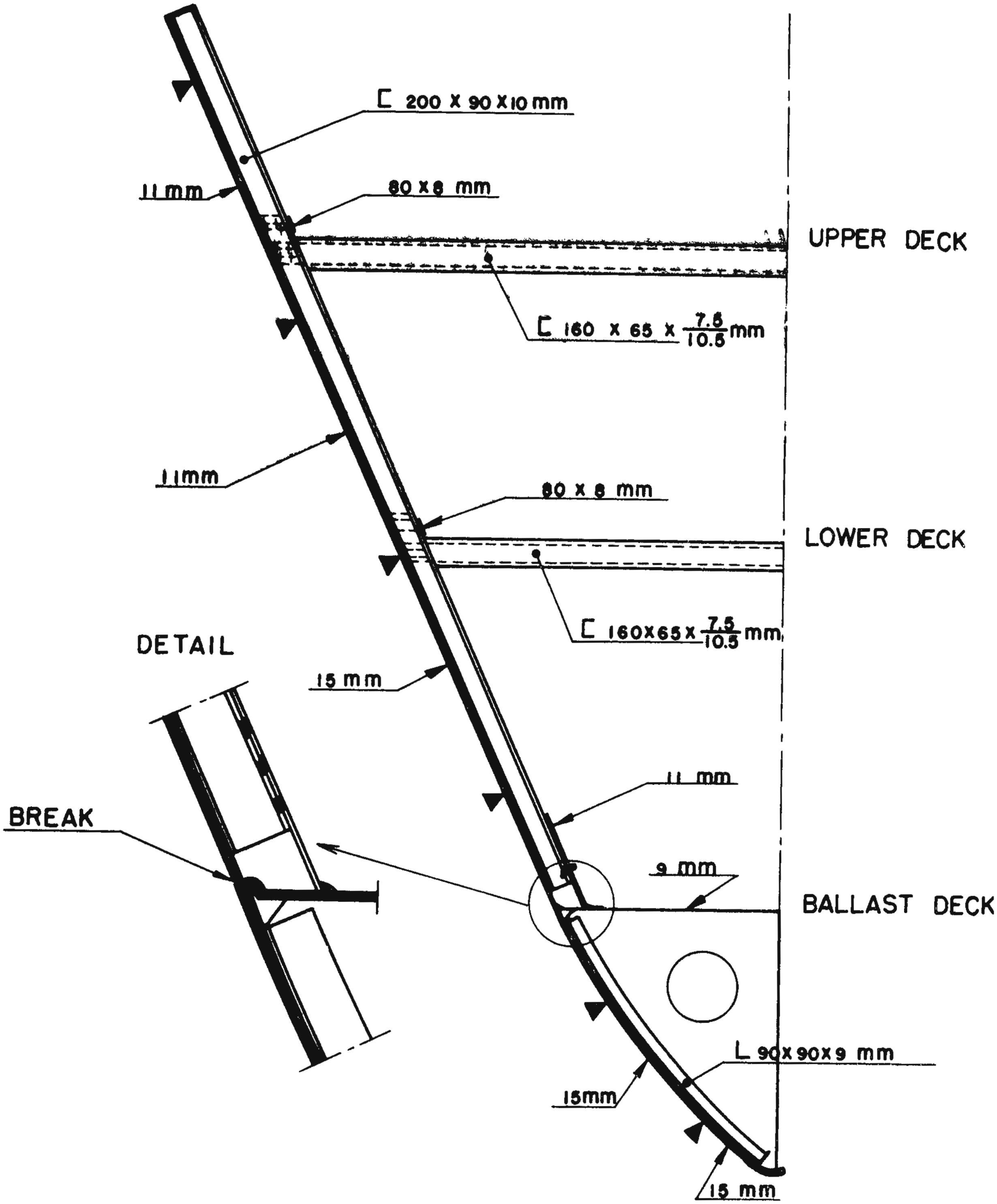
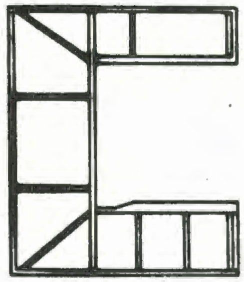
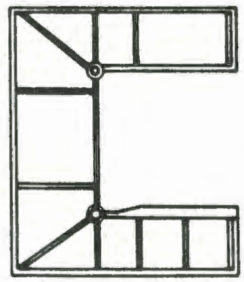


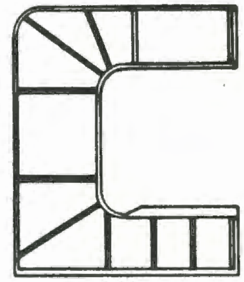
Figure 15 Banana ship. Structure in hold No. 1.
Note: The frame is interrupted when crossing the ballast tank top.



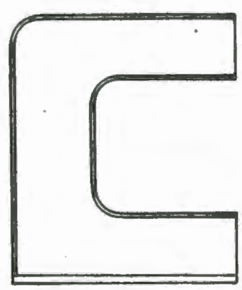
a



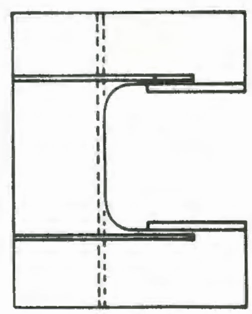
b



c



d



e

Successive test loads	P	1.5 P	1.75 P	2 P	2.25 P
Design a	2×10^6	2×10^6	7×10^4	-	-
Design b	2×10^6	7×10^5	-	-	-
Design c	2×10^6	2×10^6	2×10^6	2×10^6	1.9×10^6
Design d	2×10^6	2×10^6	1.8×10^5	-	-
Design e	2×10^6	2×10^6	2×10^6	2×10^6	1×10^6

Figure 16 Tests of various press frame designs (Weck) - Model at 1/8 - and Table giving the number of cycles before failure

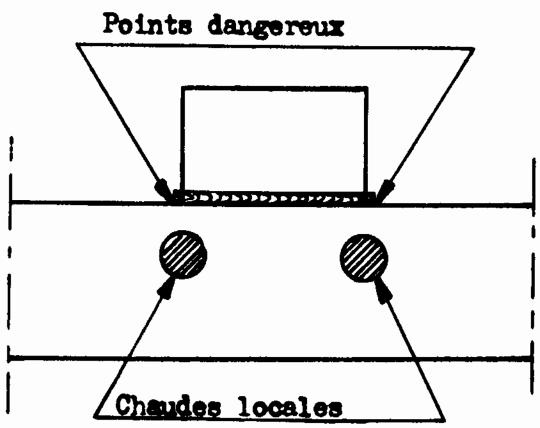


Figure 17 Example of local heating, raising compressive residual stresses at the weak points of a connection with lateral gusset plate (after Puchner). Elevation view.

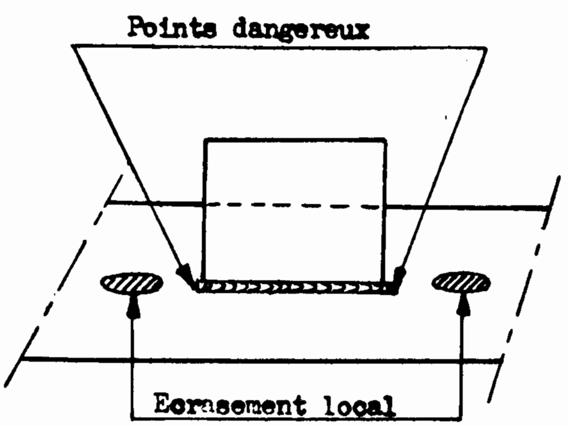


Figure 18 Example of local mechanical compression, raising compressive residual stresses at the weak points of an assembly with longitudinal attachment (after B.W.R.A.) Perspective view.

Figure 19 Multiple tool hammering of a fillet weld between welded attachment and sheet

EVALUATION OF MATERIAL TESTS FOR NOTCH TOUGHNESS

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INTRODUCTION

From the standpoint of metallurgy, I am certainly not a specialist in the field of the behaviour of welded structures. However, being involved in the general problem of the qualification of steel products and their relationships to the various requirements of steel applications, I have had an opportunity to undertake some experimental research on the widely discussed question of the rating of steels by the use of notch toughness tests. Consequently, my intention here is to propoundsome unpretentious comments on the significance of laboratory tests for assessing the brittle fracture resistance of steels.

GENERAL REMARKS

Generally speaking, it is obvious that the problem of steel selection for welded structures is somewhat different in Europe than in the U.S.; the principal reason for this situation seems to be that in Europe we are more conservative in our official standards and specifications, while at the same time the European steelmakers have put on the market a wide collection of new types of steels obtained by entirely new processes, and which are not in accordance with old specifications.

Before the war, the basic Bessemer structural steel, the so-called Thomas steel, had a great reputation of brittleness and its use was prohibited for many applications. An actual knowledge of the metallurgical factors affecting the brittleness justifies this point of view, by the analysis of the effects of nitrogen, phophorus, and also the course grain size of these kinds of rimming steels

After the war, the steelmakers developed new qualities of converter steels by means of different improvements in the steelmaking process. The problem arising from this new situation is that the steel consumer, to protect his own interests, needs to have mechanical specifications which provide for evaluation of brittle fracture resistance in order to replace the conventional and less exacting discrimination of the steelmaking process.

Everybody knows that for this purpose we actually have focused on the use of the Charpy V-notch test, and in the recommendation of the I.I.W. in this respect, the European delegates played an important part. Many considerations, many criticisms, and many discussions could be written about this peculiar choice of specimen, but this point is not the purpose of my communication. I mention it only for drawing your attention to the fact that the purpose of a mechanical test to be used for the classification of steels in a conventional order of merit is only to check whether one steel product can be regarded as good as another. Accordingly, the results of such tests generally expressed in terms of transition temperature cannot be correlated with the behaviour of the steel in the welded structure.

The only essential point is that the same metallurgical factors have the same influence on the behaviour of the steel in the test and in the construction. Although this general concept perhaps seems quite evident to you, actually many people, and particularly some steel users, tend to create much confusion when they try to correlate the transition temperature of the laboratory test with the lowest service temperature of a given steel structure.

Many examples of such mistakes could be given and among them it might be advisable to mention that some people are afraid that the service temperature of a steel structure may be lower than the Charpy V-notch transition temperature. On the other hand, I remember that recently during an international meeting a delegate criticised the Charpy V test by telling of the failure of a storage tank that occurred at a service temperature at which the Charpy V energy of the steel was about 10 kg m (72 ft lb).

It is known that the brittleness of steel is fundamentally one specific reaction of the steel, occurring in certain circumstances and consisting of cleavage rupture in a polycrystalline body. Whatever the test may be, this property cannot be directly measured, and it is to be represented by some feature of the brittleness like a drop in the rupture energy, or a crystalline appearance of the rupture, or still, the amount of fibrous rupture preceding crystalline rupture (as in the Van der Veen test).

Because these characteristics are continuous functions of the circumstances determining the brittleness, the limit must be defined by means of a criterion. Such a criterion may be of two kinds, either an arbitrary value of the variable characteristic (like 50 per cent crystallinity), or a certain value of the variable characteristic predetermined by the parameters involved in the test under consideration. An example of the latter item is the drop weight test, in which the criterion of brittleness is fixed by the prescribed deflection (this point will be discussed later). Another example is the Robertson test; in this test the variable characteristic is the transverse mean stress and the criterion is, in fact, the value of the stress at which the initiated crack stops. Owing to the test conditions, it happens that the critical stress does not vary widely with the temperature, and consequently, the criterion of brittleness does not require any arbitrary choice.

Whatever it may be, it is important to note that in any test the limit of brittleness implicates certain conventions. The property of brittleness depends on two main factors, namely temperature and stress condition.

While the temperature may be considered as a simple factor easily isolated in laboratory tests and in steel structures, the term "stress condition" does not refer only to the initial state of stress or so-called triaxiality. In my meaning, it means the entire function of the stress, including the rate of loading, the evolution of the stress before rupture, the storage of elastic energy, and so on. Generally speaking, the "stress condition" of a test should be a spectral function of the stress in relation with strain, time, and location in the body. For my communication, let us speak of <u>function</u>, which considers the stress condition as a whole.

All the brittleness tests involving the rating of steels necessarily implicate three main factors: a certain characteristic of brittleness, the temperature, and a given stress condition. The test results lead to relationships between two of these three factors; for example, the characteristic of brittleness with the stress condition, the temperature being a constant parameter. Another well-known example is the base diagram of the Schnadt theory, in which the characteristic of brittleness is the mean value of the principal stress; the stress condition is partially represented by the so-called factor, the temperature being constant. Another example of this kind will be given later.

However, in order to make easy comparison of the steel behaviour in the test and in the structure, it seems to me preferable to use the classical, and no less scientific, relationship between a given brittleness characteristic and the temperature, the stress condition being a parameter. Accordingly, every test result may provide one or more curves relating the measure of brittleness and the temperature.

A typical case is shown in Figure I by curve B. The shape of this curve is classical for the notch impact test, but it may be derived from any test. The point on the curve represents a certain value of the variable and gives the limit of brittleness in terms of transition temperature; we may assume that it represents a stress condition.

In the same manner it is possible to draw on this diagram (Figure I) the behaviour of the same steel in a given structure. But in as much as the stress conditions of a structure cannot be easily defined, we have to adopt some conventions. One interesting solution, advanced some years ago by Professor Soete, is to fix some factor in the stress condition.

The proposal is to assume that the steel assembly under question always contains a pre-existing brittle crack transverse to the direction of the principal stress. It must be noted that such an assumption is only one element of the state of stress and that it does not fix definitely the stress condition. Yet it is possible to imagine that the curve A on the diagram represents the behaviour of a given steel structure by relating the temperature to a certain variable of the brittleness, which may be usefully the principal stress, or better still a measure of the minimum plastic deformation which gives rise to brittle failure. On this curve the criterion of brittleness is automatically fixed by the maximum value to be expected for the variable in use, and consequently, it gives us the critical temperature of the structure breakdown. On the diagram, the adopted variable is the principal stress and the criterion of brittleness is therefore the maximum principal stress to be expected in the structure.

Without treating the subject exhaustively here, it may be seen that the interval "number l" represents a "confidence interval" between the lowest service temperature and the critical temperature of breakdown; such interval mainly depends on technical factors. I mean that this interval not only depends upon technical factors but mainly upon <u>non-technical</u> factors such as economy, safety factors, and importance of the structure. On the other hand, the interval "number 2" is the difference between two arbitrary transition temperatures and is dependent not only on the stress condition but equally on the respective transition criteria. This diagram is an oversimplification, but it shows briefly that the minimum service temperature of a steel structure cannot be directly correlated to the transition temperature of the test used for the rating of steels.

As an illustration for instance, the well known failures of Liberty ships may be cited. It has been found statistically that the critical rupture of breakdown of some Liberty ships was exactly in coincidence with the point of 15 ft lb observed in the Charpy V curve of the base metal. In my opinion such coincidence has no absolute significance; it shows only the relative position of the curve A (Liberty ships) and the curve B (Charpy V test).

It is quite obvious that the stress condition of many welded structures are less severe than those of the Liberty ships and therefore it is also evident that the critical temperature of breakdown of many constructions must be lower than the 15 ft lb transition temperature of the corresponding Charpy V curve.

After this general comment on the relative influence of stress conditions, I should like to bring out some particular factors relative to the stress condition which can exert a striking influence on the steel behaviour in practice. These factors are the following:

(1) The specific effect of the notch acuity.

(2) The striking influence of the thickness.

(3) Some influence arising from the use of different kinds of tests.

For this purpose, I shall refer to investigations recently carried out in Belgium.

NOTCH EFFECT IN THE NOTCH IMPACT TEST

The scope of this first investigation encompassed the following: (1) the analysis of the notch acuity influence on the entire energy transition curves when comparing different grades of steel; (2) to check whether this notch effect is the same no matter what the criterion, namely, energy absorbtion, rupture appearance, or lateral contraction.

For this purpose, we chose five different steels; among them was included a low-carbon steel plate and a high-strength structural steel in the heat-treated condition (Y.P. of 62 kg/mm^2 and T.S. of 73 kg/mm^2). From each plate were machined about one hundred test pieces similar to the Charpy test piece except for the acuity of the notch. The radii ranged from 3 mm to 0.03 mm, and in all cases the section of rupture was retained constant at 80 mm^2.

Figure 2 shows the aspect of the deepest notch acuity of 0.03 mm obtained by hand sawing with a very thin blade in a preliminary notch of 0.1 mm.

The transition curves obtained for the ordinary mild steel are shown in Figure 3. The diagram shows the energy transition curves of two steels: (1) one steel plate indicated by the letter V. The corresponding curves are the continuous lines and the figures after letter V indicate the notch radii in use; (2) the second steel plate, indicated "VN," is the same steel V but after a special overheating to obtain a course grain size. The corresponding curves are dotted lines. The lower portion of the diagram shows the variation of crystallinity measured on the broken test pieces according to standard methods.

We observe that it is necessary to keep two notch influences distinct. First, a variation of the energy level at each temperature gives rise to a general shift of the curves along the vertical axis. Second, an independent shift of the transition zone towards higher temperatures which occurs when the notch acuity is increasing. This last effect rapidly slows down when the notch radius is below 0.25 mm (Charpy V-notch radius).

The latter observation is particularly apparent on the lower diagram for the shift of the transition zone measured by the percentage of crystallinity. In this case, the transition curves are quite identical for the acuities corresponding to radii of 0.25, 0.10, and 0.03 mm.

The principal result of this diagram is to show that the real shift of the transition zone due to notch acuity cannot be expressed with accuracy by the conventional transition temperature based only on a critical energy level. On the contrary, the rise of the transition temperature corresponding to a certain crystallinity is quite representative of the real shift of the transition zone.

Figure 4 shows two similar diagrams for a heat-treated high-strength steel. Here the notch effect is quite different and is such that the transition curves suffer only a vertical shift along the axis of energy levels without any appreciable temperature displacement of the transition zone.

As is well known, the transition zone of these kinds of steel are very wide and the transition curves (energy or crystallinity) are very smooth.

Without discussing here all points of comparison that could be drawn between these two types of steel, I should only point out that the notch effect in high-strength steel mainly affects the energy absorption with much less effect on the rupture appearance.

As a by-product this diagram clearly indicates the importance of accurate machining of the Charpy V-notch in the case of high-strength steels. Consider, for instance, the conventional transition at the energy of 2 kg m. At this level, the transition temperature for the notches below 0.25 mm radius lie between 0° and -10° C., and they correspond to about 50 per cent of crystallinity.

On the contrary, when the notch is slightly larger, the conventional transition temperatures decrease rapidly in such a way that for a notch with 0.5 mm radius the transition temperature is near -80°C., while the transition temperature at 50 per cent crystallinity remains in the region of -15°C.

To summarize the remarks about the notch effect, the results from six steel plates are given in Figure 5; the figure shows the shift of the transition zone due to the notch acuity. As already noted earlier, such shift may be pictured by the 50 per cent crystallinity transition. In the figure the notch acuity is expressed in terms of stress concentration factor following Neuber's theory; the factor Kt increases with decreasing notch radius.

It is seen that the shapes of the curves are very similar and that they are practically horizontal for the sharpest notches. On the contrary, in the region of low stress concentration, the rise of the curve (just below the Charpy V-notch radius value) is different from one specimen to another. Comparing for instance the steels VN (low-carbon steel) and the steel ST (high-strength steel), the transition temperature of the first drops from +10°C. to -30°C. (40°C.), while for the other, the same effect is limited to a 10°C. drop as a whole.

Let us consider, for instance, a steel structure for which the stress condition is comparable to the notch impact test, and the concentration factor Kt is 2.8. In such a hypothetical case, the transition temperatures of the structure and of the corresponding Charpy V test should be the values of this diagram obtained by crossing on the verticals and 6, 65 respectively, with the curves of the steel. From this diagram it may be seen that when using different steels, their order of merit will be fundamentally different in the notch impact test and in the structure.

This example is fictitious, but it is obvious that such a conclusion is true, and with even greater basis, when one compares laboratory test results with arbitrary criteria to the behaviour of welded structures in which the stress conditions depend not only on the stress concentration factor, but other factors as well.

Before concluding my comments on the notch effect, I should like to point out that the behaviour of high-strength steel leads to quite different results than for low-carbon steel.

On one hand, at each temperature, the energy for initiating a crack appears very sensitive to the stress concentration factor, even in the region of temperatures where the crack is entirely brittle. In other words, the so-called brittle resistance of high-strength steel is much more dependent on notch acuity, presence of defects, etc., than it is for ordinary mild steels.

On the other hand, the notch effect alone does not appreciably affect the region of temperature in which the transition occurs.

THE THICKNESS EFFECT

After this first investigation, we carried out another one dealing with the influence of the thickness of the test piece on the transition temperature in notch impact tests. For this purpose we used test pieces like the Charpy V specimen, except for the width of the test piece. The V-notch and the length of the section of rupture were constant.

Five thick plates of about 1 in. thickness and of different types of steel were selected after a preliminary control showed that the standard Charpy V results were not affected by the sampling of test pieces along the thickness of the plates. Then, in each plate, test pieces were cut out with thickness ranging from 2 to 20 mm.

The top diagram of Figure 6 shows the effect of the width on the resilience of ship steel plate when measured at different temperatures. The x-axis is a linear scale of the width of the test piece. In this case six values were chosen from 2.5 to 18 mm, with the sections of rupture between 20 to 144 mm^2, respectively, as shown in the figure. The y-axis gives the absorbed energy related to the section of rupture unit, the so-called "resilience." The curves are for different temperatures of test. The second diagram gives corresponding values for the rupture appearance.

Considering first the two curves of resilience and crystallinity obtained at $+40^{o}C.$, we observe full ductility for each thickness and at the same time a rise of the resilience with a certain maximum being reached.

In 1924 it was reported that if the absorbed energy is related to the volume of the deformed material under the notch, the resilience should be constant for all thicknesses. This statement was based on test results in which the notch permitted large plastic strain; it seems to us that this relationship is not quite true in the case of a sharp notch and in the range of high thicknesses where the plastic deformation is not constant along the length of the notch. However, our experience does suggest that this assumption may be considered as roughly correct. When the tests are performed at lower temperatures, it is seen that the curves exhibit maximum values and at the same time, increasing percentages of crystallinity as the thickness of the test piece increases.

In a way it is possible to speak of "transition thickness" with the same meaning as the "transition temperature." It is obvious that the thickness influence is only a part of the triaxiality effect, just as is the notch acuity. Quite often it is included in the main factor "stress condition," which also includes the "rate of loading."

With regard to the brittleness factor, the stress condition may be split up in different ways. From a practical point of view it may be useful to analyse the relative importance and to separate the notch acuity, thickness, and rate of loading. In this respect it appears from Figure 6 that in the case of the V-notch, and with the rate of loading in use, the thickness of the plate plays an important role and that the thickness of the standard test piece (10 mm) lies in a critical zone.

Referring to a German publication of 1925, we can compare the rate of loading and the thickness of the plate as shown in Figure 7. This figure not only shows the absolute value of the energy absorbtion in relation to the thickness of the test piece for different temperatures, but also for three different rates of loading (static bending, 3, and 5.5 meter/sec).

At -20°C., for instance, there are three curves corresponding to the three rates of loading, but in these experiments the notch acuity was very low (like the keyhole notch). In the case of sharper notches, as in the present case, the relative influence of the same rates of loading must be even much lower.

The influence of the thickness exhibited here by the existence of sharp transition values points up the phenomenon of instability, which is undoubtedly at the root of the brittle fracture mechanism. Although it is somewhat oversimplified, Figure 8 provides a description of this instability and tends to explain the existence of the critical thickness.

It is admitted that for each state of stress there exists two types of breakdown which are related:

(1) A crystalline fracture requiring a certain amount of energy in order to be initiated in the material, or more precisely, a certain energy to be developed in a given delay time, is one type. At constant temperature this potential energy is continuously decreasing with increasing triaxiality, here represented by the thickness of the product. On the diagram this energy is shown by the curve "energie de rupture cristalline"; this curve is moving down on the diagram with decreasing temperatures or with higher rates of loading. Such a relationship is in agreement with the actual concept of the initiation of a cleavage crack in a polycrystalline structure.

(2) The ductile rupture, which leads to another relationship for energy absorbtion, is the second type. As already has been noted, this work of rupture is dependent on the plastic deformation occurring under the notch and therefore increases with thickness. The conditions of the Charpy test are such that this function is rather independent of temperature.

As long as the energy required for the initiation of a brittle crack is higher than that for ductile rupture, the test piece will undergo plastic strain and shear fracture and vice versa. On the experimental curve, the maximum value is therefore the critical thickness at which the two kinds of potential energies are equivalent.

At this point it is of interest to note that this critical value may differ from one steel to another and that such differences can occur even when the steels show the same energy curve of crystalline rupture. This observation is illustrated in the lower diagram, where two steels having two different strengths and ductilities are compared. The two steels have distinct energy curves for ductile rupture. Assuming that the crystalline rupture curve at a given temperature is the same, the dotted lines situate the critical thicknesses respectively.

Returning to the experimental results obtained on a ship steel plate, let us consider the influence of the thickness on the classical transition curves involving energy versus temperature. This is illustrated in Figure 9.

It is seen that for the different thicknesses there are simultaneously two influences. First, there is a continuous rise of the transition zone. It is of interest to observe that this transition is narrow for the low thicknesses and wide for the high thicknesses. Second, there is an independent variation of the maximum values of the curves, showing a maximum for given thicknesses.

Figure 10 concerns a high tensile steel plate of 15 mm thickness, quenched and tempered. Here the transition curves are flatter, and for the same range of thickness (2.5 mm to 14 mm) the thickness influence is much lower, particularly in the range of higher thicknesses between 7.5 mm and 14 mm, where the shift of curves is very slight.

Such behaviour of high tensile steel can be explained by the shape of the curve relating the energy for plastic rupture to the thicknesses. In the range of higher thickness, this curve is very flat because the general yielding occurring under the notch is less affected by the thickness.

It is of interest to observe, moreover, that the energy absorption (related to the section of rupture) is maximum for the thickness 5 mm, instead of 10 mm or 15 mm observed in the case of mild steels.

Considering now the cumulative influence of temperature and thickness in the notch impact test, Figure 11 shows the surfaces resulting from our previous curves. On this figure, we recognize along the temperature axis the classical transition curves; the surface has been cut in the region of the normal Charpy V specimen. We observe also the existence of critical thicknesses for tests performed at constant temperature.

Some general conclusions to be drawn from this short survey on the thickness influence may be summarized as follows:

(1) The thickness of the steel product plays an important role in the process of breakdown. Its part in the main factor "stress condition" is undoubtedly of the same kind and of the same order of magnitude as the notch effect. While the notch effect in the notch impact test becomes very slight for notch acuities below the Charpy V-notch size, the thickness of usual products greatly influences the notch toughness; however, it is obvious that there are interactions between the two effects. It seems that up to now the thickness influence has not been sufficiently taken into account when comparing the steel behaviour in the laboratory test and in the welded structure.

(2) Another conclusion is that the notch and thickness effects are less pronounced in the case of high tensile steels; this conclusion arises from the fact that both notch and thickness effects both inhibit plastic deformation. Because the critical yield point of the steel, which gives rise to ductile rupture, is absolutely higher for mild steels, such products are very sensitive to the triaxiality; in such cases, under certain conditions, the plastic yielding is inhibited by increasing triaxiality, and the energy developed in the test piece reaches the critical level necessary to initiate a brittle crack. On the contrary, the absolute value of the minimum plastic deformation determining ductile rupture is much less for high-tensile steels, and therefore such steels are less sensitive to the triaxiality.

DROP WEIGHT TEST

The last part of this paper deals with the drop weight test and its significance in comparing different tests results, particularly in analyzing the specific influence of stress conditions. During the past few years, many comparisons of this kind have been published in the technical literature: for example, the recent work of Johnson and Stout of Lehigh University in a paper dated July, 1960.

The published papers show that a rough comparison between transition temperatures obtained by different tests has no meaning, due to the variety of criteria, their arbitrary definition, and above all the differences in stress conditions.

However, such studies have shown that if the interpretation is made carefully, it is possible to draw the respective influences of metallurgical factors and external factors such as stress condition. As a result, statistical comparisons between Charpy V and drop weight tests permit us to observe some trends due to the state of stress.

The drop weight test is well known, and consists of a limited bending of a plate in which a running brittle crack has been previously initiated.

Figures 12 and 13 show the equipment in use in our laboratory, including the weight and the rails and the test piece on the anvil after the test. In Figure 14 the principal details of the test are shown.

According to Pellini and Puzak, the scope of the test is to measure the ability of the steel plate to display at the tip of the running crack sufficient ductility to stop it. The criterion of success is quite simple and is based on the rupture appearance. The nil ductility transition is the maximum temperature at which the brittle crack is running through the full width of the test piece.

Referring to our experience of the test applied during 3 years on about one thousand steel heats and steel plates from 10 mm up to 100 mm in thickness, we have analyzed in detail the parameters of the drop weight test.

As far as a metallurgical test for the rating of steels, the drop weight test is, in principle, very convenient, and practical advantages are attached to it.

Among these advantages are the following:

(1) The transition temperature between ductile and brittle behaviour is very clean, the interval being generally less than 10°C.

(2) The test results are fairly independent of the initial energy of the blow and the rate of loading.

(3) Unexpectedly, the test result is affected only slightly by the direction of the crack in the steel specimen.

(4) Under restricted conditions, the results do not depend on the amount of prescribed deflection.

(5) The N.D.T. is practically independent of the test-piece dimension (except the thickness) and of the weld bead and its notch.

(6) Finally, the procedure is very simple and does not require special care or accurate equipment.

Against these advantages, the drop weight results are greatly affected by the thickness of the plate. Such influence is strong and surpasses many other factors.

An example is shown in Figure 15. On this diagram the crosses represent N.D.T. values of four steel plates of different thicknesses (15, 20, 25 and 30 mm) but coming from the same heat. For comparison the solid dot points are the N.D.T. values obtained on the same steel heat by cutting down test pieces in the thickest plate of 30 mm. In this case subsize specimens have been used for the thicknesses 5, 10, 15 mm, in accordance with standard procedures. For thicknesses of 15, 20 and 25 mm we have used standard specimens as would be made for rolled thicknesses.

It is quite clear that a pure geometrical effect of the thickness does account for all the N.D.T. variation. It is of interest to note that this thickness effect on the N.D.T. cannot be balanced by an appropriate deflection of the test piece. It seems quite likely that the thickness of the plate and the length between the supports of the test piece are interferring in the stress condition by a compound effect, in addition to the influence of thickness on the local plastic deformation.

In my opinion, the principal reason is that the elastic energy stored in the material by the external loading differs widely for different thicknesses and this elastic energy cannot be affected by a slight modification in the deflection.

This opinion is confirmed by some data obtained in our laboratory and also available in the literature, which shows that when the thickness is great (above 75 mm or 3 inches), the absolute value of the N.D.T. is nearly equal to the Robertson arrest temperature. Such occurrence is not surprising because, as a matter of fact, the N.D.T. is a measure of resistance to the crack propagation.

Owing to this assumption, comparison between Charpy V and drop weight results may be of interest. In Figure 16 the top diagram shows four typical Charpy V curves of steel belonging to the classes B, C, and D. On each curve, the data represents the Charpy V energy corresponding to the nil-ductility temperature. In accordance with the observations of many other people, this energy level increases as the weldability of steel increases.

Broadly speaking, if we assume that the Charpy V-notch transition temperature measures resistance to initiation of a crack, while the N.D.T. expresses a certain resistance to crack propagation, this observation leads to the conclusion that for the steels of the best class (D), the resistance to initiation is relatively higher than their resistance against propagation. This opinion is, moreover, confirmed by a similar comparison between Charpy V and Robertson results.

The lower diagram in Figure 16 shows statistical results of the same kind: frequency curves of transition temperature (on one hand Charpy V 20 ft lb--on the other hand N.D.T.) concerning 20 heats of class D and 15 heats of class C. The possible influence of thickness has been avoided by choosing all steel plate thicknesses between 15 and 20 mm. It is seen that the mean N.D.T. values agrees with the Charpy V 20 ft lb in the case of class C steel. On the contrary, for the class D these values are significantly different.

However, another important factor is the difference in scatter bands, which are more important for the Charpy V transition temperature, and particularly for the class D steel. Although no detailed discussion of this point is given here, this difference can be explained by the metallurgical factors which accompany the different weldability classes (killing, normalization, controlled rolling, etc.).

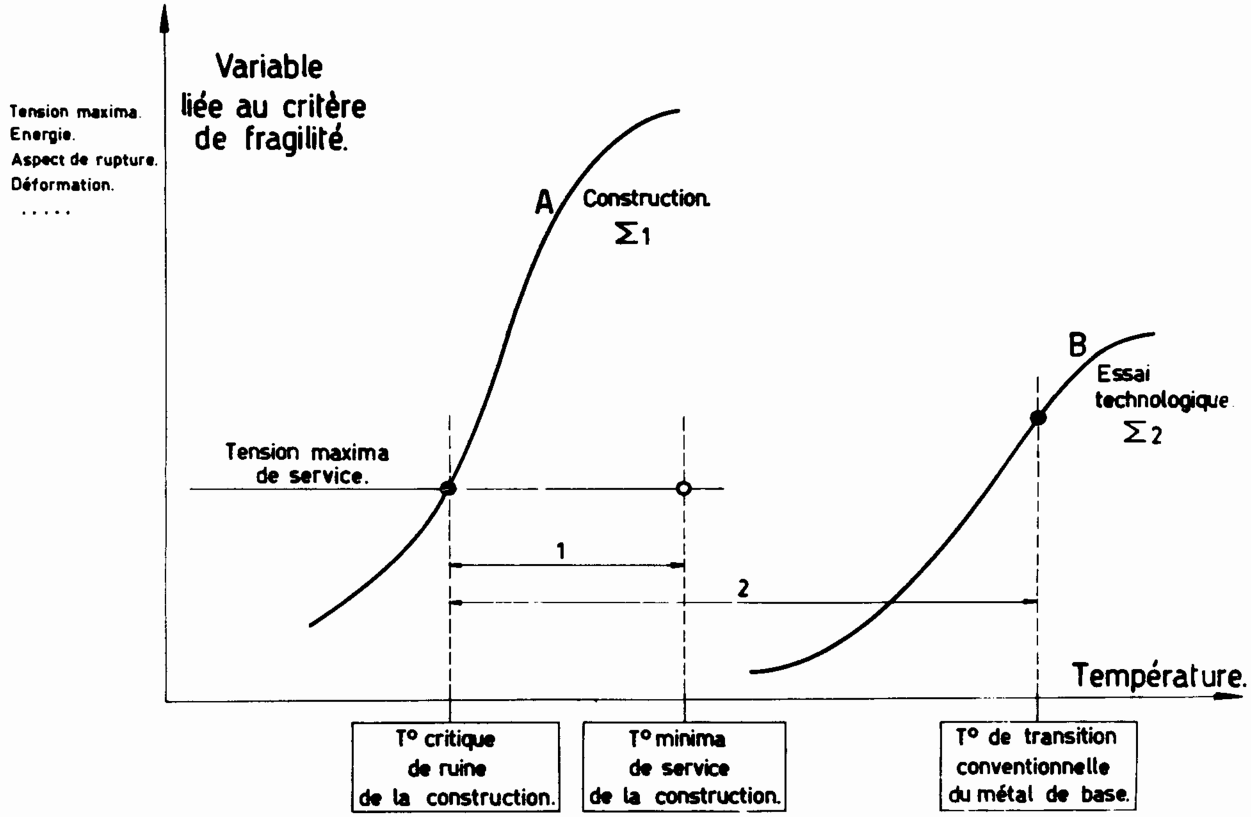


Figure 1 Diagrammatic representation of brittleness of steel in the test and in the construction.

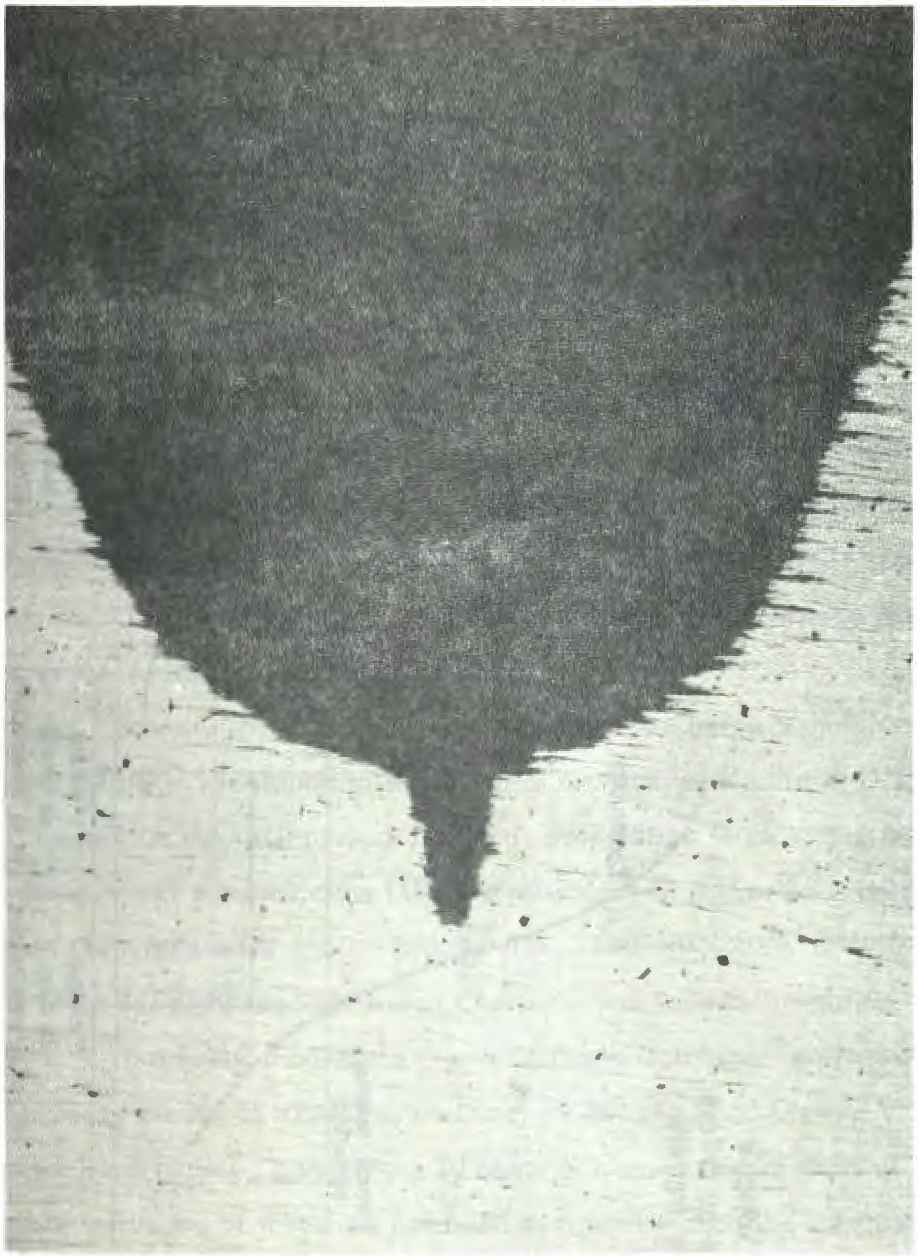
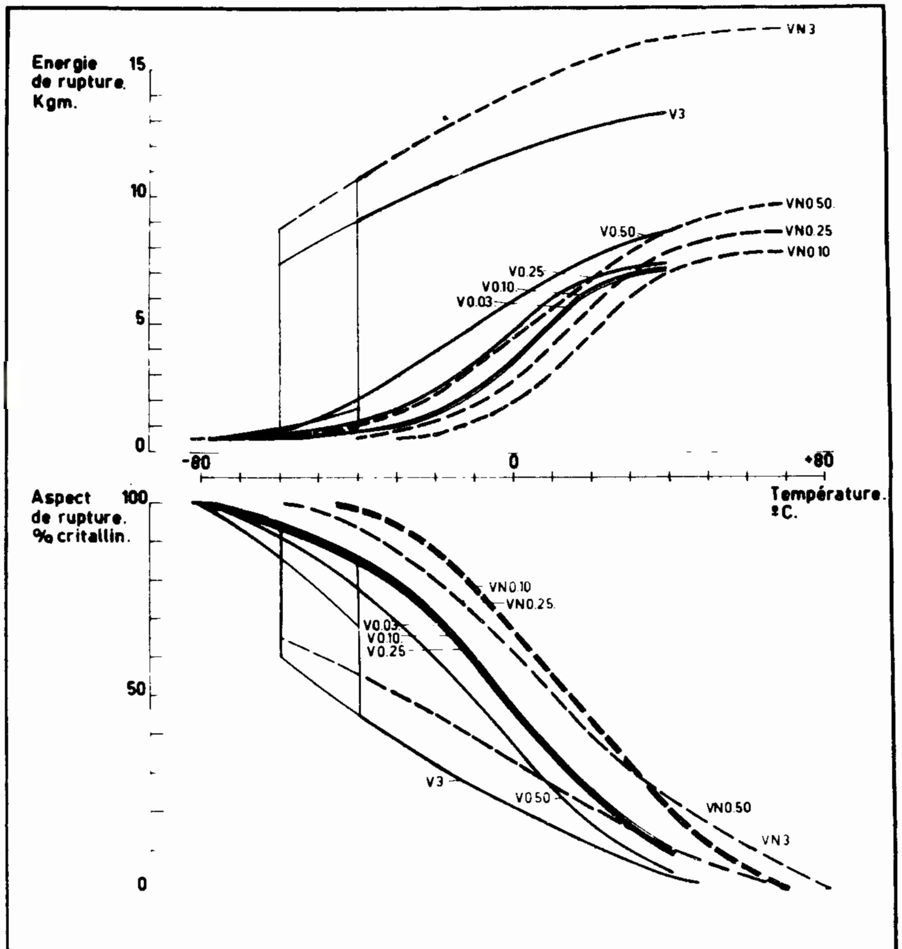


Figure 2 Notch acuity of 0.03 mm- by hand sawing in a preliminary notch of 0.1 mm.



CNRM.Lg.	Courbes de transition à rayon d'entaille variable. Acier V Acier VN.	1958

Figure 3 Transition curves obtained by the use of different notch acuities- Steels V and V.N. (The figures represent the notch radii in millimeters.)

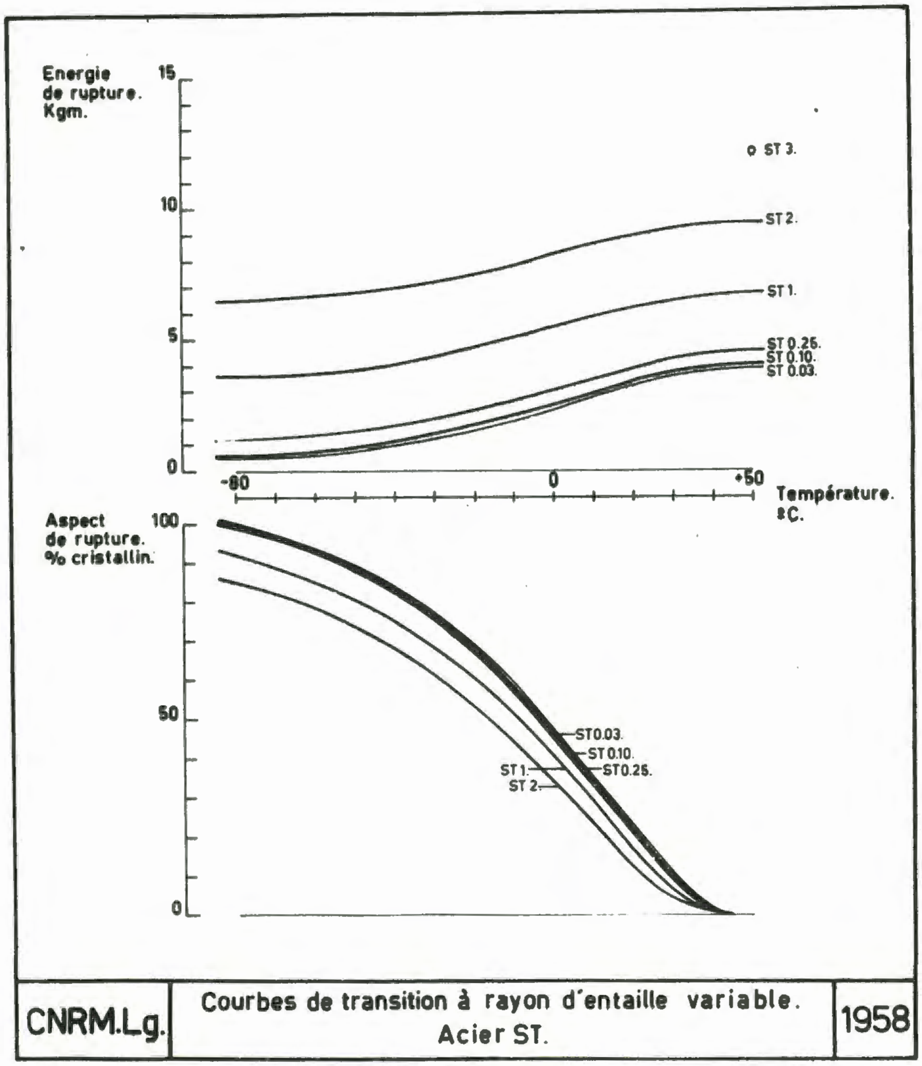


Figure 4 Transition curves obtained by the use of different notch acuities. High tensile: S T. (The figures represent the notch radii in millimeters.)

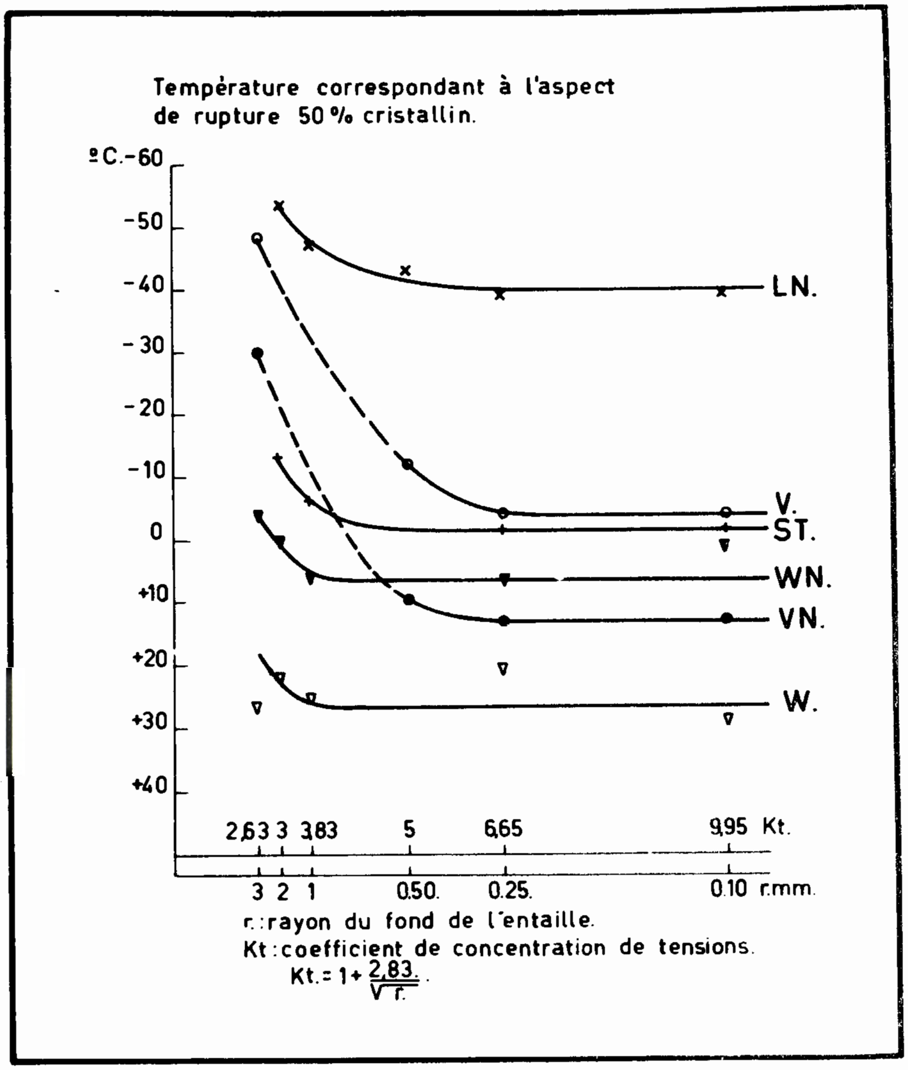
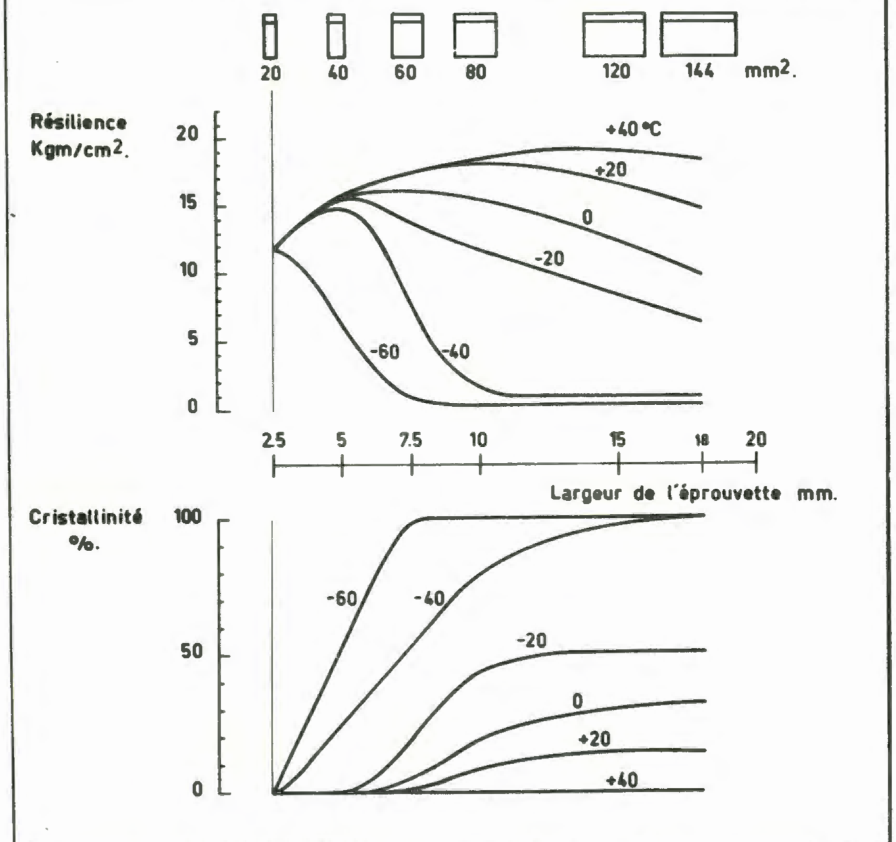


Figure 5 Effect of notch sharpness on the transition temperatures (50 per cent of crystallinity) of different steels.



Acier de construction 42/50 Kg/mm 2. Tôle de 20 mm.
Fonctions Résilience et Aspect de rupture–Largeur de l'éprouvette.

Figure 6 Structural steel 42/50 kg/mm^2 - 20 mm plate. Effect of width of specimen on the energy absorbtion and crystallinity, at different temperatures.

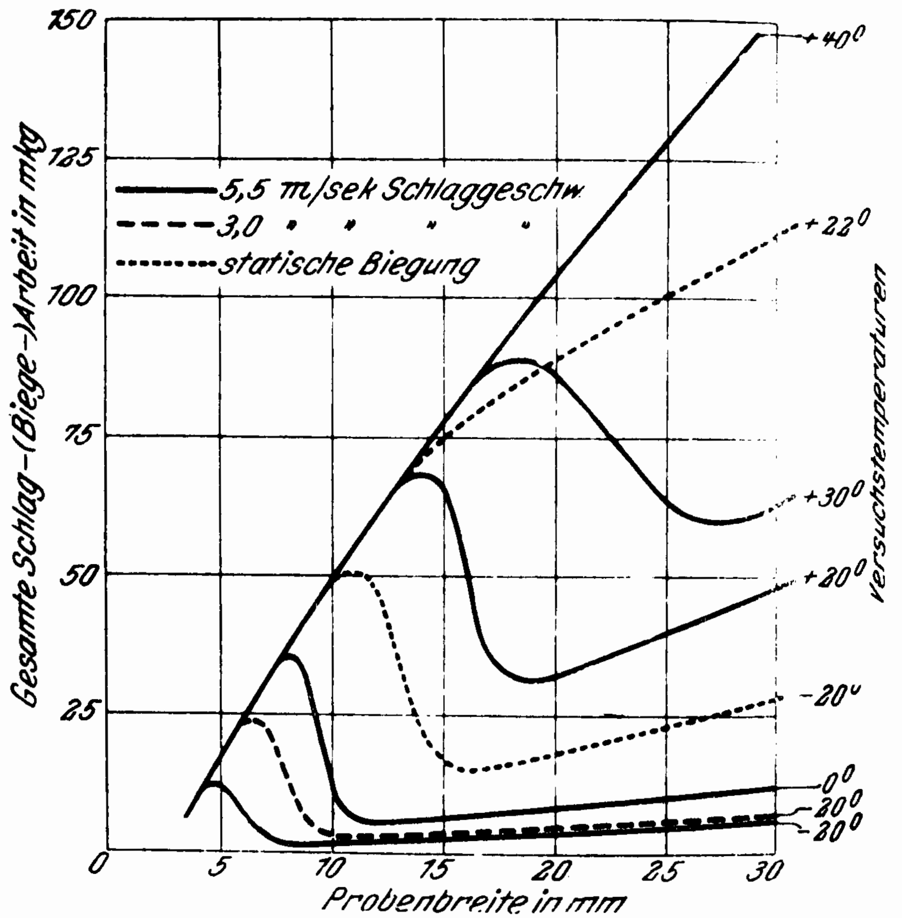


Figure 7 Comparative influence of testing temperature and impact velocity on the "Energy versus specimen" relationship (after Mailander)

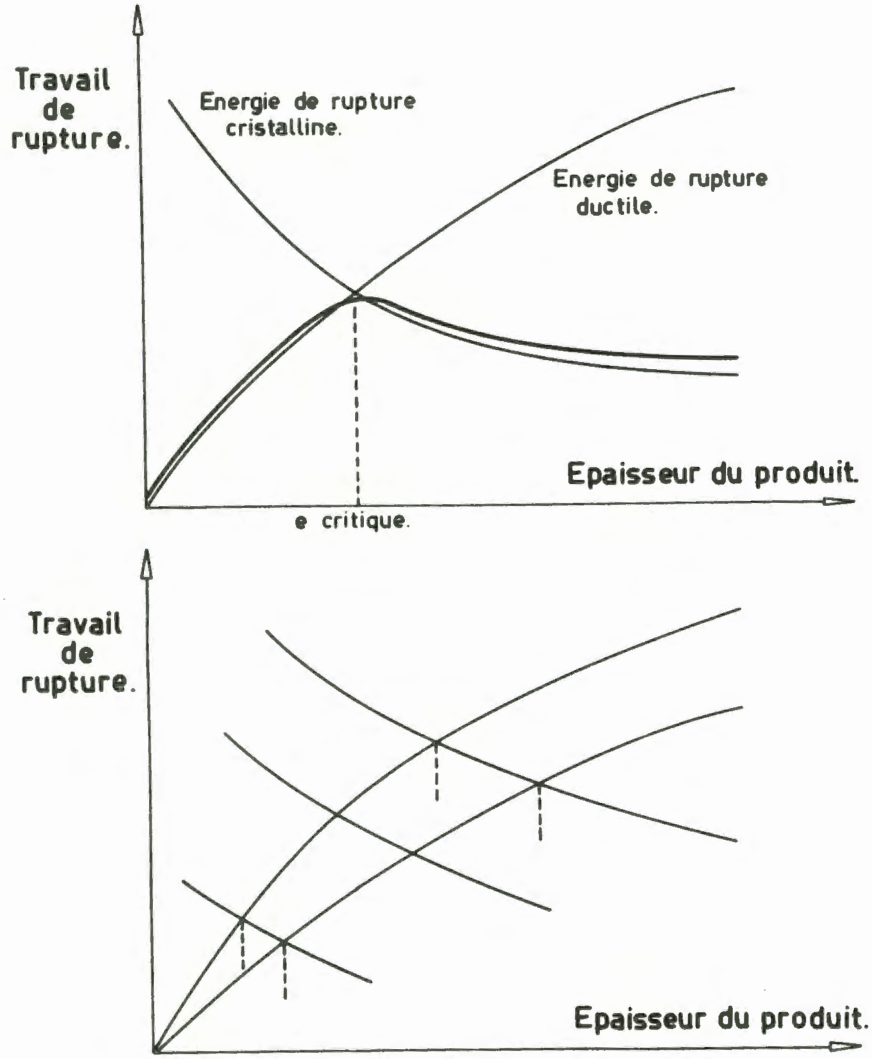


Figure 8 Diagrammatic representation of the effect of thickness of the plate upon the energy absorbtion in the rupture by impact. Existence of a critical thickness giving rise to brittle fracture at constant temperature.

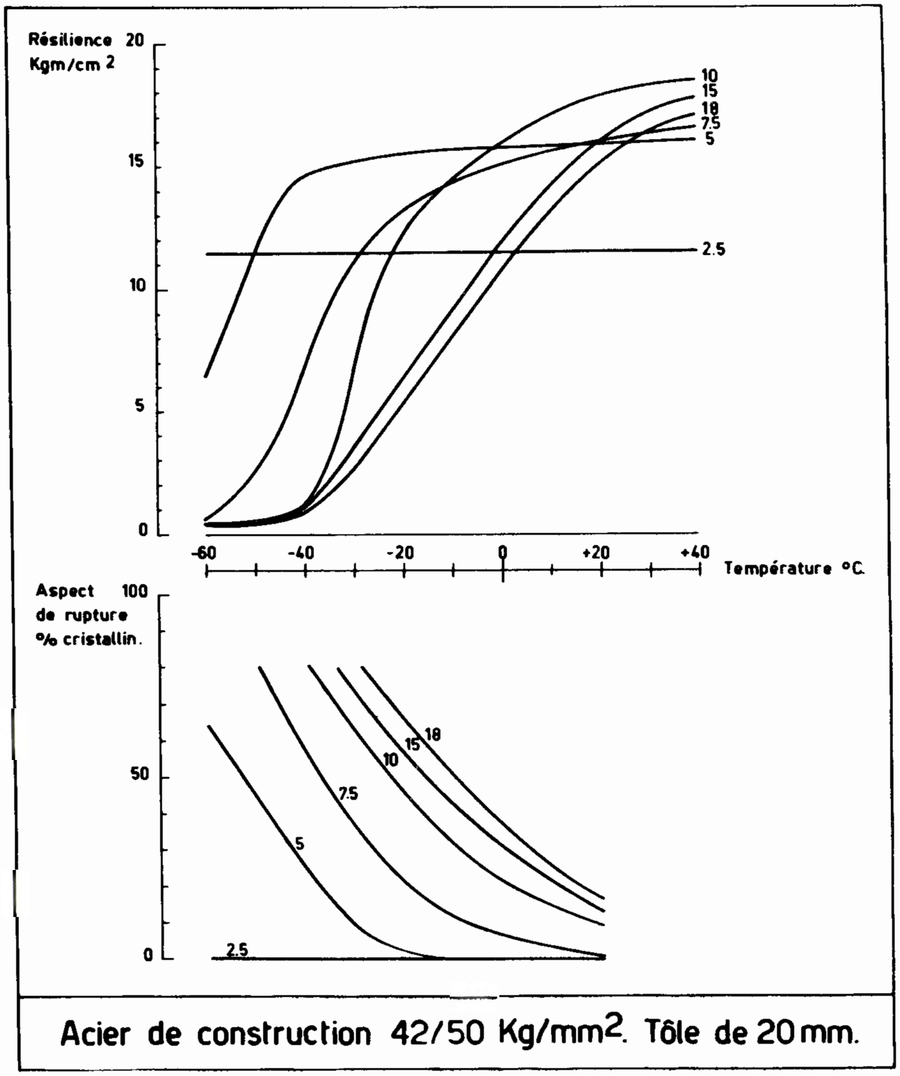
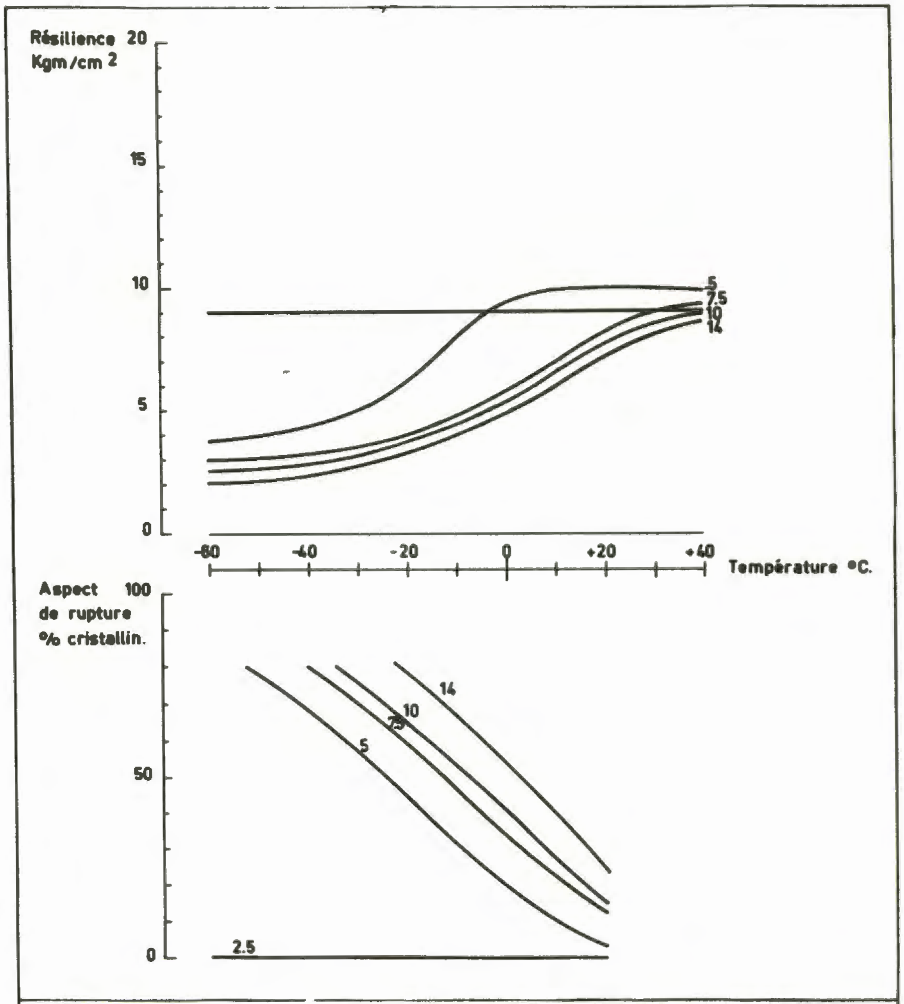


Figure 9 Effect of specimen width on the impact transition of mild steel. The figures indicate the width of test pieces in millimeters.



Acier à haute résistance – état traité. Tôle de 15 mm.

Figure 10 Effect of specimen width on the impact transition of high tensile steel.

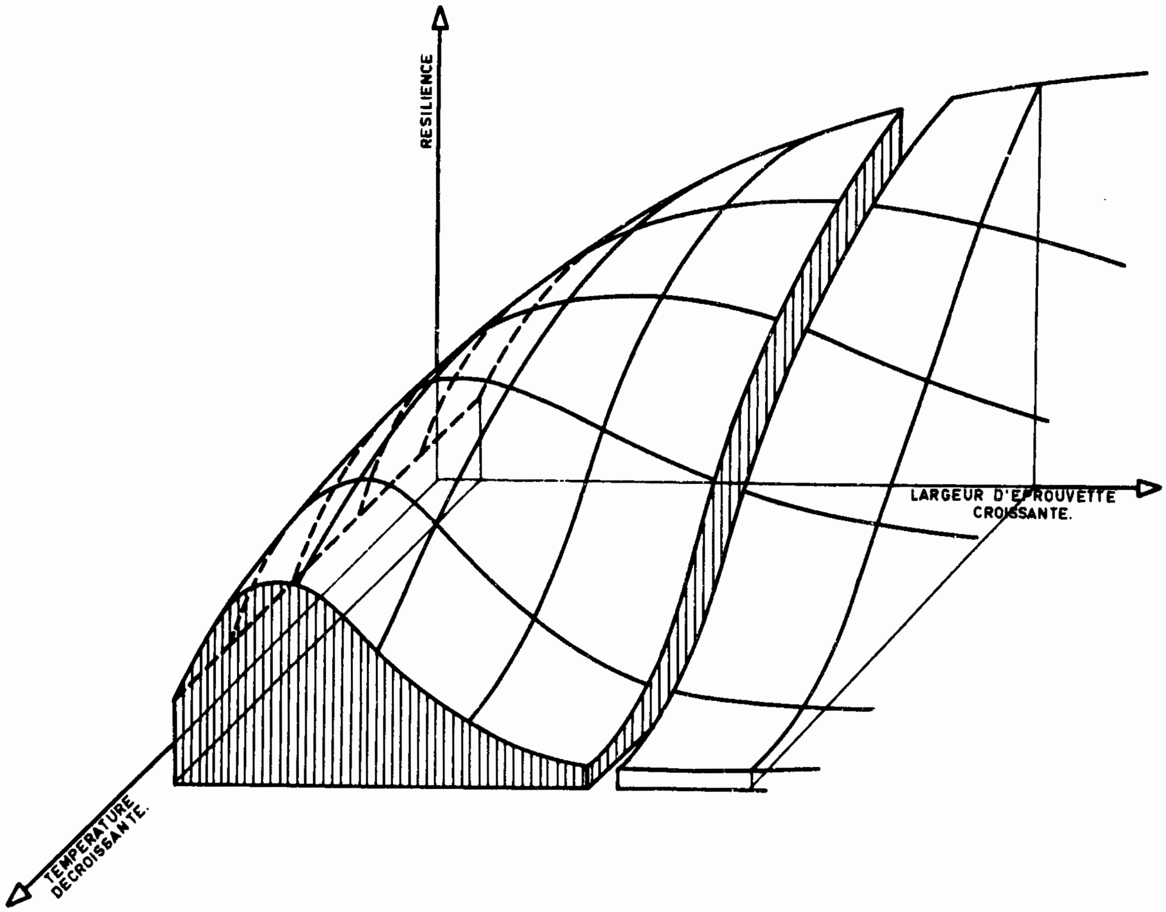


Figure 11 Diagrammatic representation of the effects of width and temperature in the impact test.

Figure 12 Drop weight test equipment - weight and rails - height 7m.

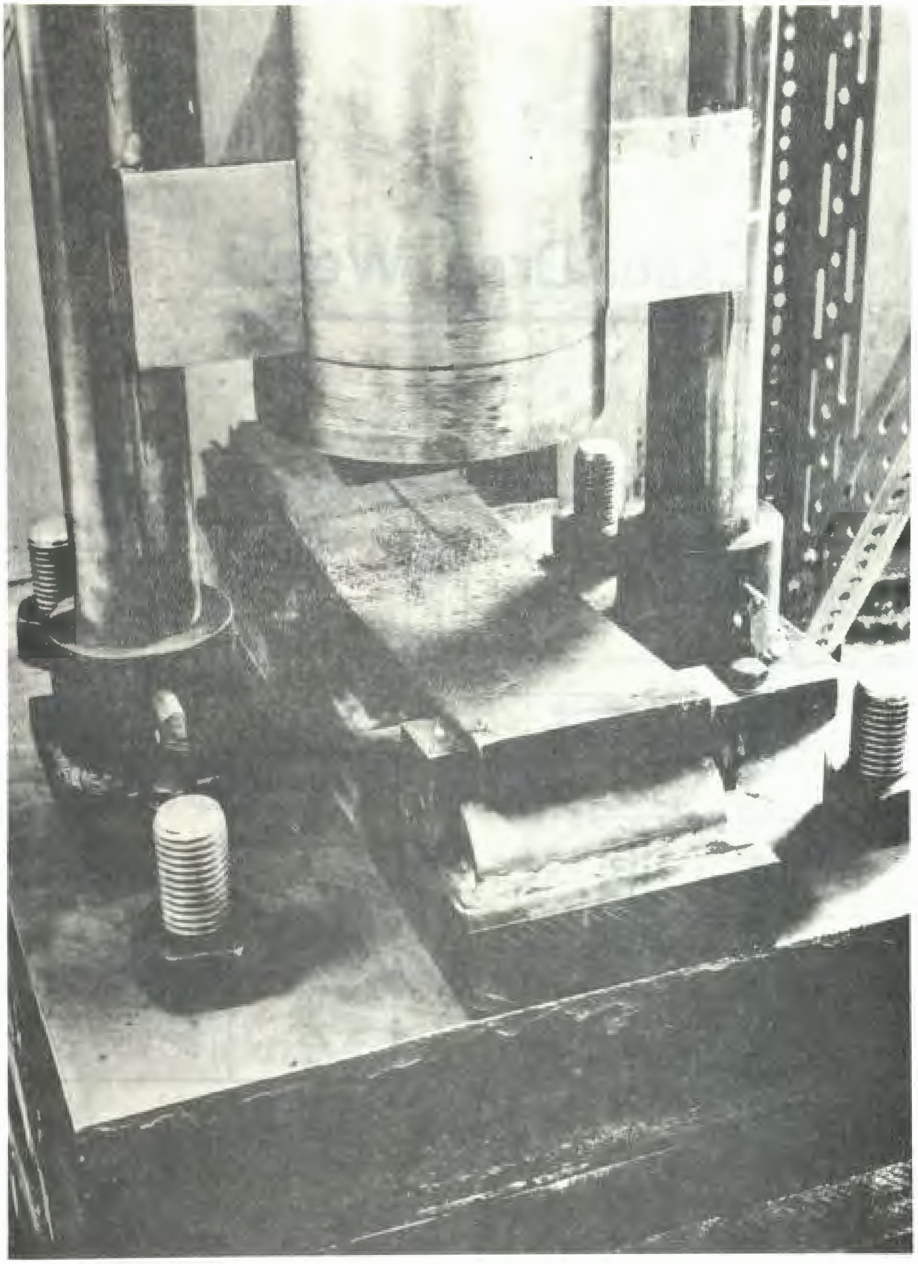
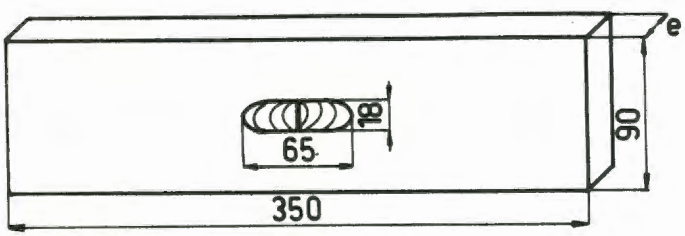
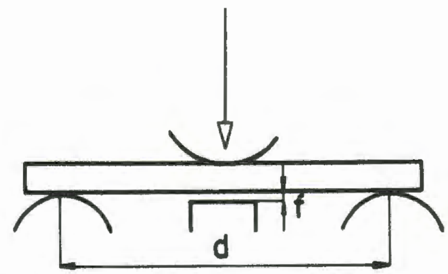


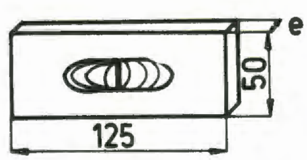
Figure 13   Drop weight test equipment: the anvil.

Essai Drop Weight.





Eprouvette normale (e≥18 mm.)



Eprouvette réduite (e<18 mm.)

Epaisseur mm.	Eprouvette.	d mm.	f mm.
≤ 13	réduite.	100	2.3
≥ 16	réduite.	100	1.9
18≤ e <25	normale.	300	7.6
> 25	normale.	300	5.1

Figure 14 Main characteristics of the N.R.L. drop weight test.

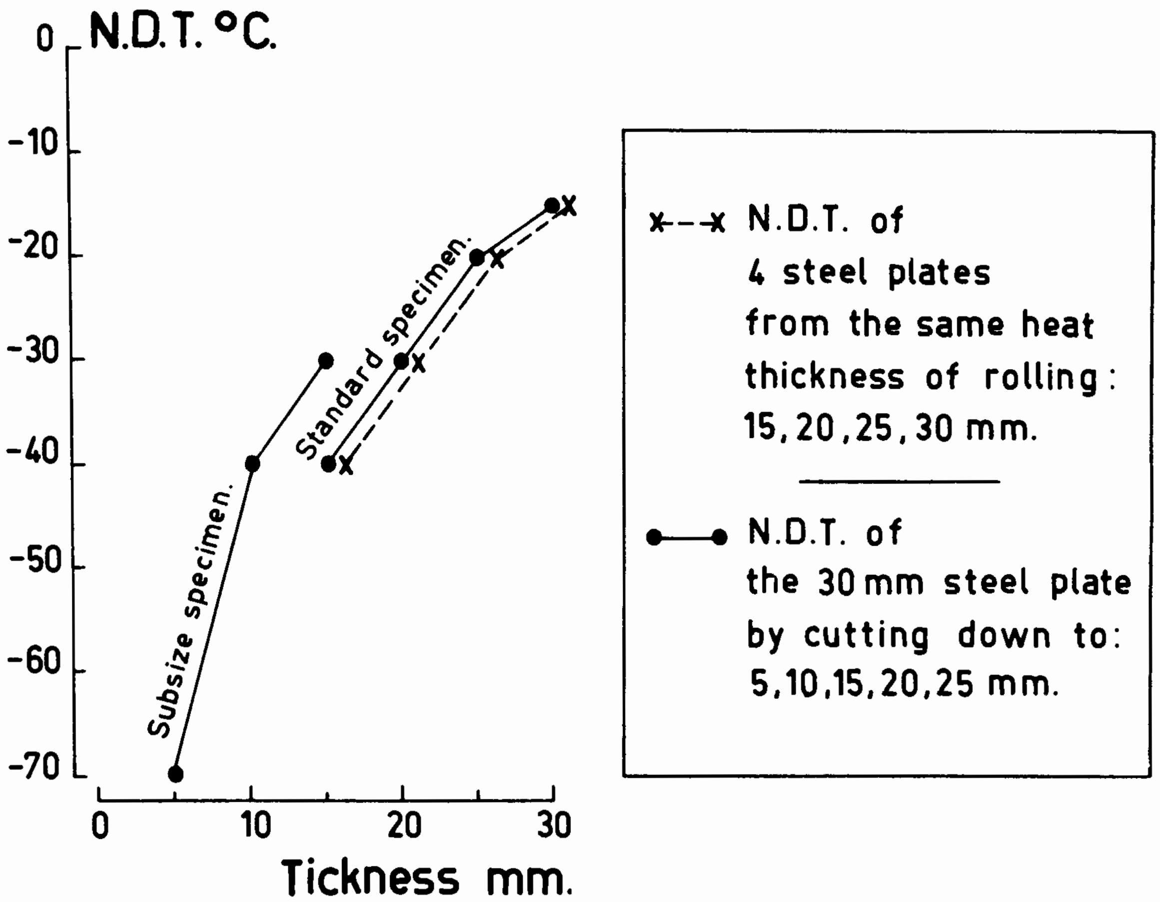
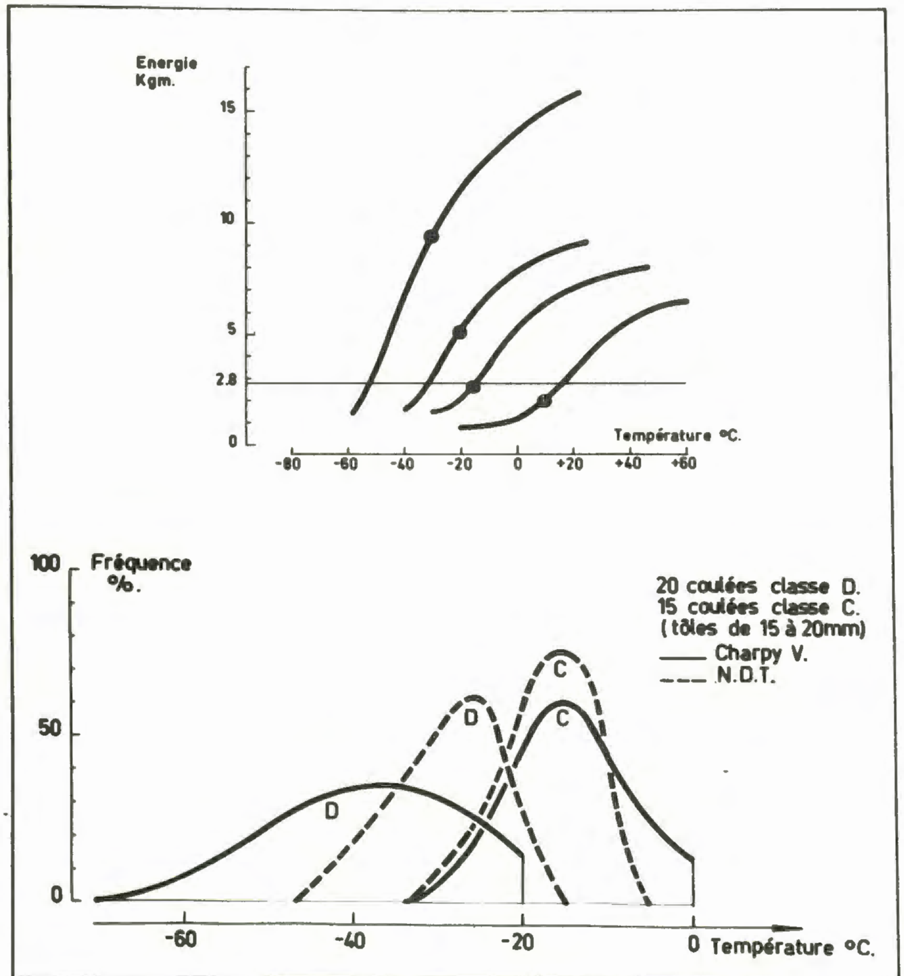


Figure 15 Effect of specimen thickness on the nil ductility transition in the drop weight test.



T° de transition $\boxed{\text{Charpy V. 2.8kgm.}}$ et $\boxed{\text{N.D.T.}}$

Aciers de construction: classes C et D.

Figure 16(a) Typical Charpy V transition curves on steels of weldability classes B, C, D, (the points refer to the corresponding N.D.T.)

(b) Frequency curves of transition temperatures
20 heats of steels class D.
15 heats of steels class C.
Continuous lines: Charpy V 20 ft lb
dotted lines: N.D.T. drop weight test.

THE SUSCEPTIBILITY OF WELDMENTS TO BRITTLE FRACTURE

A.A. Wells, British Welding Research Association , Cambridge, England

INTRODUCTION

You will easily understand that we are more interested at British Welding Research Association in the effects on brittle fracture in all manner of fabrication operation operations, than in the steel-making side. When I was first introduced to the subject many notched wide-plate tests had been performed in this country, and a proportion of them at this laboratory (University of Illinois). I was impressed at the time by the ability of all these plates to reach general yielding before fracture, irrespective of testing temperature and notch sharpness. It was suspected that such large stresses before fracture had not been sustained in a number of casualties. However, ship structures are complicated and one could not be certain about the influence of stress concentrations. It was at about this time, we had in the U.K. the two failures of oil storage tanks at Fawley.

There are probably few welded structures of simpler conception than the cylindrical oil storage tank. The hoop stress is precisely known and there are few attachments to disturb the stresses. Here, therefore, was proof that casualty fractures arose at low applied stresses even without impact conditions.

The Fawley fractures were both nucleated at circumferential welds where there had been defects. This suggested again the possibility of interference from longitudinal weld residual stresses, and one was reminded of the experiments of T. W. Greene in this country.

Greene performed bend tests on plates with central butt welds where saw-cut defects had been placed in the prepared edges for welding. A feature of his results common to the storage-tank fractures was the occurrence at low applied stresses of arrested brittle cracks across the butt welds. It therefore seemed imperative to repeat Greene's work with tension specimens. Large loads would be needed, but T. S. Robertson showed how these could be achieved in special-purpose test rigs, and one of 600-ton capacity was built. We were later to add a 2000-ton rig, capable of testing 3-in. thick plates, and I should like today to show you the results of several years' work with this equipment.

TESTS ON FIVE STEELS

Early work, in which Greene's results were substantially repeated on a brittle steel, permitted the artificial saw-cut defect at a longitudinal butt weld to be standardised. Remarkable increases of fracture strength were also achieved by thermally stress relieving the test plates (also shown by Greene), and by simply prestretching the as-welded plates before testing, at a temperature above the cleavage transition. However, it is also of interest to see how the different classes of available mild steels reacted in this test in the "as-welded" condition, by means of which such an encouraging comparison with casualty conditions seemed to have been achieved. Of the five types P was a standard semi-killed product, Q had more manganese and less carbon, while R, S, and T were all fully killed and normalised, being made to fine grain practice. The results for steel Q alone are shown in Figure 1, since almost all the variations of possible behaviour can here be demonstrated.

Some specimens were welded with manual electrodes having a rutile coating while others had a low hydrogen coating. In some specimens the arrested type of short fracture was experienced, while in others the fracture passed through the plate at one instant. These features are distinguished in Figure 1 where the lengths of short fracture, and the plastic extensions of specimens strong enough to be yielded before fracture are all shown. It will be noticed that there is an upper temperature limit for low-stress fracture and another at somewhat higher temperature for transition from cleavage to shear fracture. At low temperatures there also appears to be a limiting stress below which through fracture does not occur.

In these two respects a comparison with the Robertson crack arrest test results (for which the line is superimposed) is encouraging. The principal difference is that through fractures in the welded and notched wide plates occur at somewhat lower stresses, and there are two notable specimens where the fractures at very low stresses were arrested only by a curious influence at the edges of the specimens.

After the occurrence of arrested short cracks it was found in every one of the large number of tests so affected, that re-initiation of fracture required an applied load sufficient to yield the whole of the remaining area of cross section. As is well known, many tests such as these have now been performed in Japan, where there has been a similar experience with re-initiation. There must also be some exceptions, however, and it has been found there that re-initiation is relatively easier in some brittle steels.

The trend of results described for steel Q above was repeated in the other steels. There were some differences, notably that in one of the fully killed and normalised steels, S, very few low-stress fractures could be produced, even below the Robertson crack arrest temperature. These features appear in Figures 2 and 3.

COMPARISON WITH SMALL-SCALE TESTS

At first sight the correlations of Figure 2 between the fracture appearance transition temperatures for the welded and notched wide plates and the transition temperatures determined for each steel in four other small-scale tests do not appear encouraging over the range of five steels. In fact, the Robertson crack arrest and Pellini drop weight transition temperatures appear to be the most closely associated with wide-plate behaviour. This may not be surprising, since in the one case the crack arresting conditions in the Robertson and wide-plate testing specimens are similar. In the second case both the welded and notched wide plate and the Pellini drop weight specimens each contain a notched weld deposit as crack starter.

With regard to the Charpy V-notch and Tipper notched tensile comparisons there are more variations to be accounted for. The very large energy absorption of steel R at the temperature for low-stress fracture in the wide-plate test is a noticeable feature, as is the tendency for all the small-scale test transition temperatures to be high for steel P.

These differences remained unaccounted for until Dr. Tipper became interested in the behaviour of the five steels following a straining and aging treatment. While I cannot associate Dr. Tipper with the interpretations that follow, it is now widely recognised that these materials are influenced by welding in a manner comparable with strain ageing.

INTERPRETATIONS

When the long butt weld is made in a wide plate, the heated material is not allowed to expand or contract by the full amount during the thermal cycle, and plastic strain takes place instead. It may be postulated that the combination of plastic straining and heating will cause ageing in the familiar manner. Strain ageing is known to raise the transition temperature. This idea is supported by the results of tests where 2-mm deep saw cuts are made in the prepared edges for welding in a wide plate so that parent-metal Charpy specimens can be extracted after welding with the 2-mm deep saw cuts in place of the standard V-notch. Such specimens fail in the impact test with a much reduced energy absorption. In order to compare the results of Charpy tests on strained and aged material with those of the wide plate tests, a plot was made as in Figure 3. The vertical ordinate represents the fraction of tests in a given sample of wide-plate tests, where fracture strengths were below yield point. For the 60 tests it was found to be convenient to choose sampling temperature intervals of about 20° F. When compared with the relevant Charpy properties appropriate to each temperature interval, the correlation with the strain-aged condition is clearly much better than for the as-received condition, and impact energy also takes precedence over fracture appearance. In fact, it is seen that the impact energies for given low-stress fracture probabilities are in order of the yield points of the steels. With regard to fracture initiation this correlation promises to remove some of the difficulties evident in Figure 2. It is also noteworthy how the response to strain ageing varies even among fully killed and normalised steels.

Another feature that arose in the wide-plate tests was a tendency for the specimens welded with rutile coated electrodes to show easier initiation, that is lower fracture strengths, than those specimens welded with low hydrogen electrodes. Associated with this was the observation that nearly all the rutile weld deposits showed oxidised pre-cracking across the saw-cut plane, whereas most of the low-hydrogen deposits did not. It is thus plausible that the lower fracture strengths were simply caused by larger initial defects. This feature has long been suspected and is demonstrated far more explicitly in tests of 3-in. thick specimens where the range of possible defect size was much greater.

THICK PLATE TESTS

The risk of brittle fracture is a consideration during welded fabrication from thick plates, even if structures such as reactor pressure vessels operate subsequently at temperatures well above the transition. It is for this reason that recent attention at B.W.R.A. has been concentrated on fracture tests in 3-in. thick normalised silicon-killed boiler plate. The tests were performed in a similar manner to that described earlier, with saw-cut artificial defects in the prepared edges for longitudinal multi-run manual butt welds. Tests were performed in both the as-welded and stress-relieved conditions, with work on the significance of defects confined to the stress-relieved plates.

An interesting feature of the tests on as-welded specimens was the comparison involving notches placed either near the plate surfaces or at the plate centres. It was found that the latter type of defect did not easily produce low-stress fractures in comparison with the former. This was traced to a residual-stress effect; as the weld beads are laid on one another they contract during cooling, and so partially relax the contraction tensile residual stress in the runs beneath. It was demonstrated by measuring the dimensional relaxation of a welded joint cut into longitudinal strips near the weld that the tensile residual stress had effectively vanished near the middle of the plate. As would be expected, this phenomenon was absent in the stress-relieved plates, and the combination of some surface saw cuts, some central saw cuts and other artificial defects extended by oxidised pre-cracking permitted the influence of defect size on fracture strength to be assessed. The result is shown in Figure 4 where logarithmic co-ordinates have been used to describe both defect length in the plane of the plate and fracture strength. It will be seen that for defects of less length than the plate thickness there is a tendency for fracture strength to lie between the yield and ultimate strength for the material, whereas for longer defects (represented by arrested brittle cracks in as welded specimens) the fracture strength lies close to the yield point. Observed strengths with the smaller defects at given temperatures also appear to satisfy the Griffith criterion of fracture strength inversely proportional to the square root of crack length. Thus, the logarithmic plot is justified and the inset has been prepared of Irwin's K_{Ic}--the stress intensity for fracture initiation--plotted against temperature. This quantity is numerically equal to $\sigma\sqrt{\pi a}$, where σ is fracture stress and a is the half-length of the defect. There is surprisingly little scatter in this curve. It will be noted that the correlation does not embrace defect sizes greater than the plate thickness, meaning that defects substantially reaching the plate surfaces are excluded. It is possible that the influence of reduced triaxiality, seen in the growth of ductile shear lips when the fracture reaches the plate edges, may be associated with this behaviour.

There is no time left to discuss further reasons to explain these observations, but they encourage the belief that the influence of defects on brittle fracture in welded structures may yet be quantitatively predicted.

I hope that I may have indicated how brittle-fracture research can be of assistance in an expanding welding technology.

There followed a 15 minute film on the conduct of tension tests on 3-in. thick notched and welded steel plates in a 2000-ton equipment. The following references have since been added.

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2. A. A. Wells, "Brittle Fracture Strengths of Notched 3-in. Thick Steel Plates," British Welding Journal, August 1961.

3. A. A. Wells, "Welded Ferritic Steel Construction for Intermediate Low Temperature Service," A.S.T.M., Atlantic City Conference, June 1961, Paper 80c.

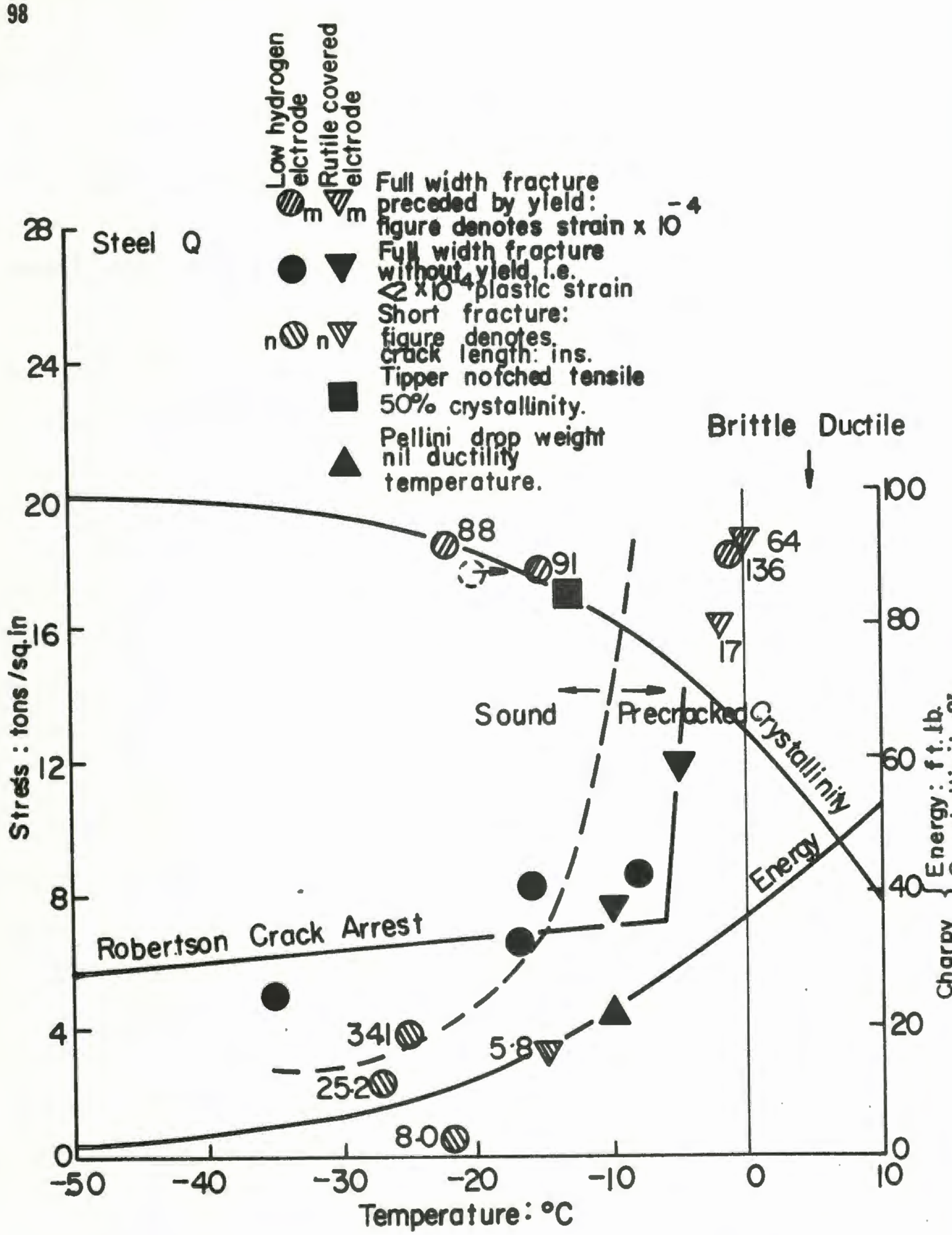

Figure 1 Fracture strengths of notched and welded wide-plate specimens of steel Q at various temperatures with associated Charpy, Pellini, and Robertson test data.

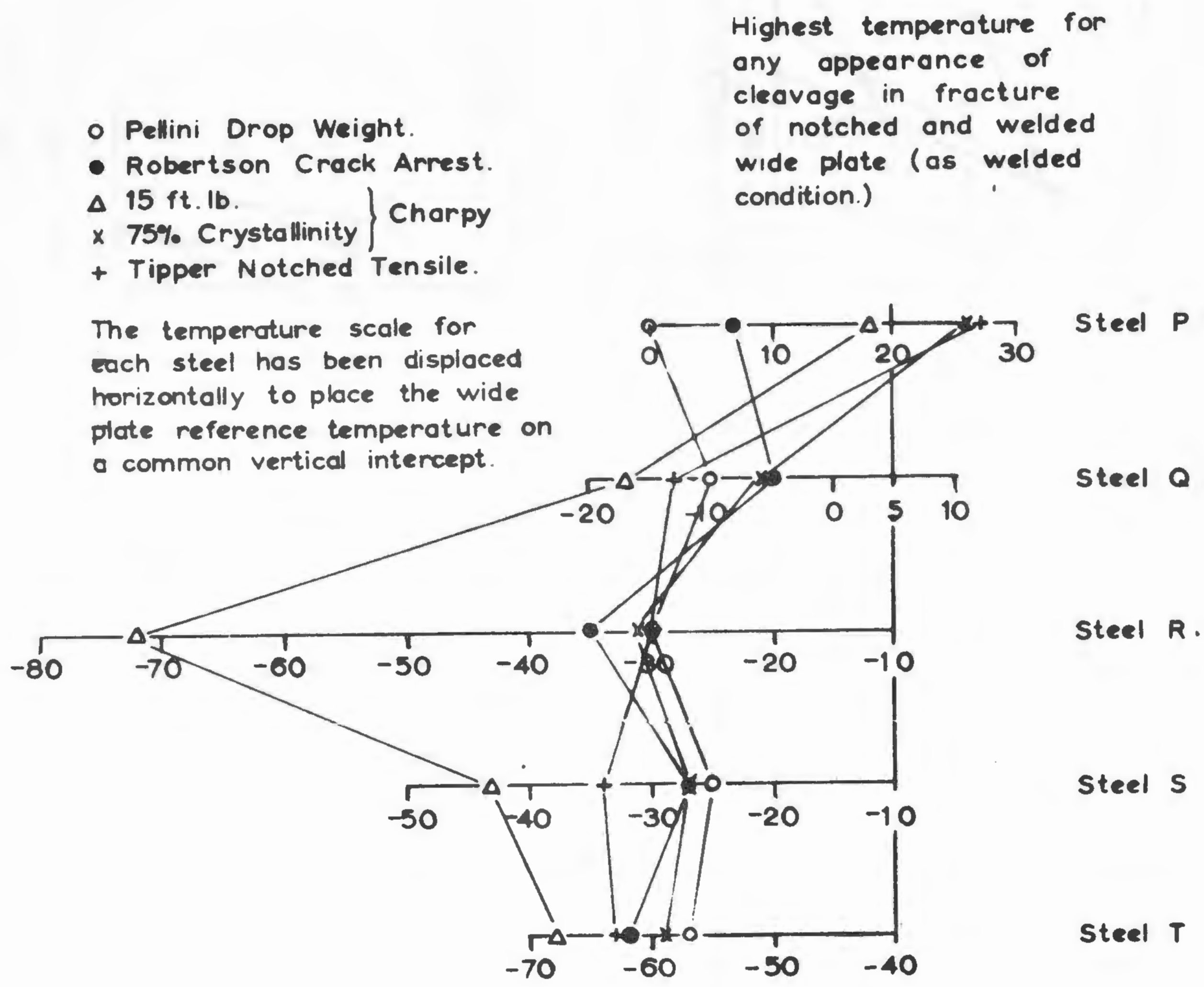

Figure 2 Transitional crack-initiation characteristics of welded and notched wide plates compared for five steels with small-scale transition-temperature test results.

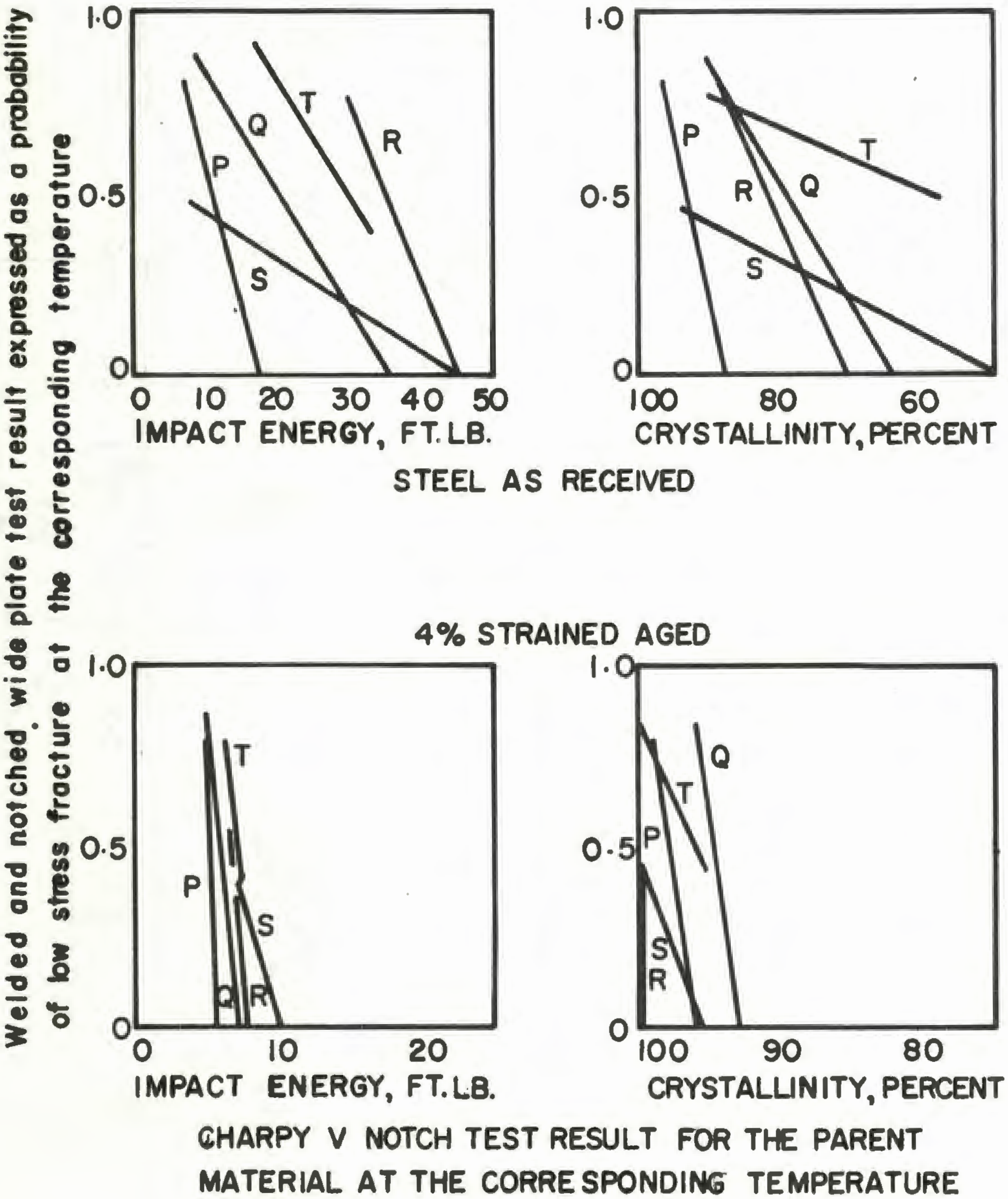

Figure 3 Brittle-fracture initiation in notched and welded wide plates compared with Charpy impact tests of as-received and strain-aged parent material.

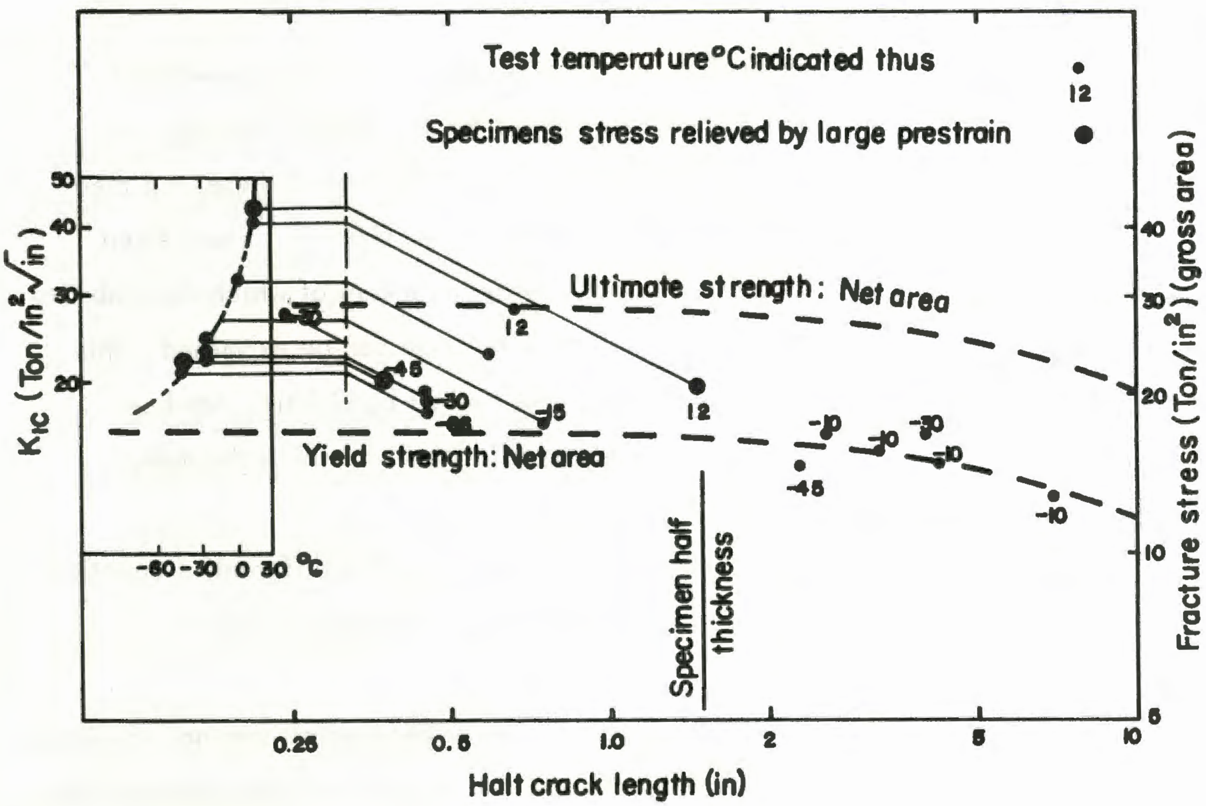

Figure 4 The influences of defect size and testing temperature upon fracture strengths for thermally stress-relieved specimens of 3-in.-thick plate.

REFLECTIONS ON FRACTURE MECHANICS

G.M. Boyd, Lloyds Register of Shipping, London, England

INTRODUCTION

Fracture mechanics, which is the study of the phenomenon of fracture in terms of mechanical concepts such as force, velocity, energy, etc., has in recent years advanced along two main lines. The first of these, the theoretical approach, first applied to this problem by A. A. Griffith,[1] has relied mainly on the principle of conservation of energy, by means of which the stability of the mechanical system formed by the fracturing body can be examined. This approach, though apparently simple in the case treated by Griffith, has been found somewhat complicated when applied to mild steel, to which the modern effort has been mainly directed.

The greatest difficulties lie in setting up the differential equation on a realistic basis, and in measuring the quantities which enter into the equation.[2]

The second line of attack has been experimental, and has consisted mainly in causing fractures in plate specimens, and measuring loads, surface strains, and velocities, with the object of fitting these observed quantities into a theory which would enable predictions to be made.[3,4,5,6,7,8]

Although monumental and extremely valuable experimental work has been done, particularly by active groups in the U.K., U.S.A., Japan[9,10] and other countries,[11] a satisfactory bridge between observation and prediction has not yet been constructed. Yet the need for such a bridge is vital, and until it can be constructed, the experimental and theoretical efforts can only be regarded as exploratory. We are groping in a dark jungle, hoping that somehow the light will break through.

The present notes will provide little illumination, but it is hoped that they may clear away a small part of the confused undergrowth, and point to possible paths. It will be necessary to re-examine very critically some of the fundamental concepts in common use as a realistic prelude to a more constructive treatment.

THE PHENOMENON

The breakdown and rupture of mild steel is an exceedingly complicated process.[12] When a body of this material is subjected to loading, it may respond in many different ways, and the problem with which we are confronted is to predict this response, given the initial configuration of the body and the applied loading.

It is important to appreciate here that only in a few very simple cases can we predetermine either the loading or the stresses. These are determined by the response of the body, and this response depends not only on the configurations mentioned, but also upon unknown properties of the material itself.

We are in the habit of ignoring this important fact, and we talk of "applying" certain loads and stresses. If the loading consists of weights attached to the body we can with some confidence regard them as predetermined, provided the body can sustain them without important changes in the configurations. But in practice the loading is usually applied by hydraulic forces through a spring-like mechanism, the testing machine, the relevant characteristics of which are often unknown.

The stresses are even more subjective and unpredictable, since they are simply the intensities of the reaction forces which arise within the body in order to balance the external loads. The stresses and their distribution depend entirely upon the response of the material to the loading, i.e., upon what we call the "properties" of the material.

These properties are only very imperfectly defined, usually by simple tensile or compression tests, and are commonly extrapolated beyond their validity by means of insufficiently developed theories, such as the theory of plasticity. It must be remembered that we are considering the final breakdown and fracture of the metal, where the fundamental assumptions of elasticity and plasticity theory, such as isotropy, homogeneity, and continuity are no longer valid.

During the process of breakdown, the stress-strain relationships are radically and permanently changed by slip, work hardening, precipitation, migration of constituents, and the formation of micro-fissures.

ENERGY CONCEPTS

The concept of energy is compounded of the concepts of force, distance, and time. It is therefore an omnibus quantity which is of little direct use to the engineer concerned only with loads and stresses. Nevertheless, the use of energy concepts can be illuminating because they enable the application of well established principles, such as that of conservation of energy and that of least action, which must be satisfied by any mechanical system. Once it has been established that a theoretical treatment satisfies these principles, we may proceed to analyse the energy into its components.

The principle of conservation of energy states that in any mechanical system the total energy is constant, except insofar as it is affected by energy supplied from or to the surroundings. If no energy crosses the boundaries of the system, the total energy is constant, regardless of any changes within the system.

The principle of least action states that when a mechanical system passes from one state to another, it does so by a path for which the integral of energy multiplied by time, i.e., the "action," is an absolute minimum consistent with the constraints.

In the case we are considering, i.e., the breakdown and rupture of a body of mild steel, these two principles must be satisfied at all stages, and on all scales from the atomic to full size, apart from some considerations arising from relativity theory which need not concern us here.

In considering breakdown and fracture it is useful to concentrate attention on a small element of material in the path of the main fracture. Such a small element will pass through all the stages, i.e., elastic strain, plastic flow, shearing, micro-fissuring and fracture. During the first two of these stages a certain amount of elastic potential energy will build up within the element. This energy is potentially available for conversion into other forms. If the element remains continuous, the elastic energy will continue to increase, but at some stage micro-fissuring or fracture will occur, and the potential energy will be converted into surface energy and kinetic energy. Some of the latter may be reconverted to elastic potential energy in surrounding parts of the body.

If the energy released by the micro-fissuring or fracture of an element is sufficient, it may lead to further micro-fissuring or fracture of surrounding elements, and the fracturing may become catastrophic. This is the condition we know as "unstable" fracturing.

In view of these considerations, it is clearly of great interest to study the factors which contribute to a high intensity of locally stored elastic potential energy, and it is convenient to do this by means of the stress-strain or stress-flow relationships.

STRESS-FLOW RELATIONSHIPS

For illustration purposes we may consider the idealised diagram shown in Figure I in which the true tensile stress on an elementary cube is supposedly plotted against the "flow", i.e., the elongation of the element. This quantity, the flow, is usually called the strain or natural strain, but it is thought preferable here to use the term flow, to distinguish it from elastic strain.

This diagram indicates the stress associated with plastic flow at each stage of flow. The elastic strain associated with the same stage of flow is a much smaller quantity, obtained by dividing the flow stress by E, the modulus of elasticity. The elastic energy per unit volume stored in the element at any stage of flow is represented by the area of the shaded triangle in the diagram, i.e.,

$$u = S^2/2E$$

where S is the stress associated with the particular stage of flow. Note that it is independent of the flow, and depends only on the stress.

We may now consider the effects of u of some of the main factors, i.e., triaxiality, temperature, and rate of straining.

TRIAXIALITY

For an elementary cube, oriented in the direction of the principle stresses, it is customary to describe the stress-flow relationship in terms of the octahedral shear stress, T, and octahedral shear strain[13] or flow, Y, in the form:

$$T = AY^n \tag{1}$$

where

$$T = 1/3 \sqrt{(x-y)^2 + (y-z)^2 + (z-x)^2}$$

in which x, y, z are the principal stresses and

$$Y = 2/3 \sqrt{(p-q)^2 + (q-r)^2 + (r-p)^2} \tag{2}$$

in which p, q, and r are the strains in the x, y, and z directions respectively. A and n are constants.[14]

If now we introduce the notation

$$\mathrm{II} = \sqrt{1 + a^2 + b^2 - a - b - ab} \tag{3}$$

in which $a = y/x$ and $b = z/x$, we may write Equation 1 in the form:

$$x = \frac{3A}{\sqrt{2}\,\mathrm{II}} Y^n \tag{4}$$

The quantity II, as defined by Schnadt[15] (Equation 3) is a measure of what is conventionally known as triaxiality. II varies from zero for fully triaxial condition ($x = y = z$) to $\sqrt{3}$, i.e., 1.732 for torsion.

It can be seen from Equation 4, therefore, that increasing triaxiality increases the steepness of the stress-flow diagram, as shown in Figure 2.

The stored elastic energy per unit volume at a given flow value varies as the square of the stress, so that the following comparison results for Project Steel A (SSC.19,)[17] for flow of 0.20 at 25° C.:

for pure shear,	II =	3,	u = 30 in lb/cu in
" simple tension,	II =	1,	u = 90 " "
" restrained tension,	II =	$\frac{1}{2}$,	u = 360 " "

It can be seen that the kind of triaxiality which can realistically be expected to develop within a body near the tip of an active fracture (II = $\frac{1}{2}$) can increase the stored energy, as compared with simple tension, by a factor of 4. (Note: Schnadt proposes II = $\sqrt{3}/4$ = .433 for a crack tip).

It is important to recall here that the simple relationship of Equation I is only valid when straining is continuous, i.e., when there is no reversal or other important change in the form of loading. It has been shown[16] that for work hardening materials there is no general one-to-one relationship between stress and plastic strain. The relationship depends on the strain history.

It is also important to recall that the triaxiality at fracture may be very different from that which exists initially, while the behaviour is truly elastic. The true triaxiality (or II value) at rupture cannot therefore be predetermined, nor indeed can it be measured.

TEMPERATURE

The effects of temperature on the stress-flow relationships for mild steels have been specially studied by Klier, et al[17] and some of their typical results for Project Steel A are re-plotted in Figure 3. It is seen that the main effect of reducing the temperature is to increase the slope of the flow-stress curve, which has the effect of increasing the stress, and therefore the stored energy at a given stage of flow. This is more clearly shown in Figure 4, in which the stored energy per unit volume is plotted against flow (natural strain).

The effect of reducing temperature is therefore very similar to that of triaxiality, namely to increase the locally stored elastic energy at a given stage of flow.

Over the practical range of values, the effect of temperature is less marked than that of triaxiality.

	Practical range of values	Factor by which stored energy is increased
Triaxiality	II = 1 to II = $\frac{1}{2}$	4
Temperature	+ 50° to - 50° C.	1.5

These results of course apply only to this particular sample of steel and are given here purely for illustration purposes.

RATE OF STRAINING

The study of the effects of strain rates is complex, partly due to the subjectiveness of strain, mentioned earlier, and partly due to the ambiguities in the literature as to the real meaning of "rate of strain." The subjectiveness means that definite rates of strain cannot be imposed experimentally since the actual rate which develops is not a simple function of the rate of application of the load. The ambiguities arise from failure to distinguish clearly between the rate at which the load is increased, say in lb/sq in/sec and the duration of the load, i.e., the time in seconds during which the load is maintained constant at a given value. The terms "speed of loading" and "rate of strain" are therefore, by themselves, by no means explicit.

Fortunately, however, the general effects of high rates of load application are fairly clear. They tend to increase the steepness of the stress-flow curve, in much the same way that has been shown for decreasing temperature and increasing triaxiality.

This is shown by Figures 5 and 6, taken from a paper by Clark and Wood.[18] These are stress-strain curves for tensile tests made at varying rates of load application. Figure 5 is for a 0.19 per cent carbon annealed steel, while Figure 6 is for a Type 302 stainless steel. It is to be noted that the strain scales of the two diagrams are different. The annealed carbon steel shows a definite and pronounced yield-point, while the stainless steel does not. It is clear from this data that increasing the rate of load application increases the steepness of the stress-flow curve, i.e., increases the stress corresponding to a given state of flow.

Over the range of loading rates studied in these experiments, and at the small flow values recorded, the effect of speed is to increase the stresses at a given strain by about 20 per cent, so that the locally stored energy is increased by a factor of about 1.4. In an actual fast fracture the factor may be greater than this.

The effect is further complicated by a "delayed yield" effect, which was observed in the carbon steel, but not in the stainless steel. In the former, yielding did not occur immediately upon the application of a load, but only after a finite time which decreases with increasing load.[19] The delay time was of the order of 0.2 to 0.7 sec.

This delayed yield effect appears to be superimposed upon the general raising of the flow stress, and has a very pronounced effect on the stresses at low strains, as seen in Figure 5. For example, at 1 per cent strain, the stress is raised by about one-third, so that the locally stored energy would be increased by a factor of about 1.8 or nearly doubled.

It is of interest here to note that steels which have a definite yield-point are notoriously more liable to brittle fracture than those which have no definite yield-point. This may be conjectured to be at least partly due to the delayed yield effect observed in the former but not in the latter.

Wood and Clark[34] and Krafft[35] have shown that the delay time increases with falling temperature. This is brought out clearly in a diagram by Barrett[36] (Figure 17), from which it is deduced that over the temperature range from $+50^\circ$ C. to -50° C. (the "practical range") the delay time increases from about 10^{-4} to 10^{-1} sec, i.e., by a factor of about 1000, at an arbitrary stress level of 60,000 psi in annealed 0.17 per cent carbon steel.

The delayed yield effect operates mainly at small strains, when the structure of the metal is probably very little altered by plastic flow. The effect may therefore be an important factor in fractures which occur with very little deformation.

Summarising the foregoing, it appears that triaxiality, high rates of loading and low temperature all operate towards increasing the locally stored elastic energy. They are also known to be associated with brittleness, and it might be supposed that the brittleness is due to the increase in stored energy. There is, however, no direct evidence for this, and the remark should be merely regarded as a pointer which might with advantage be followed up.

It is not known whether the effects of triaxiality, high rate of loading and low temperature are additive; if they were, it is probable that their combined effect over the practical range of values could increase the stored energy by a factor of about 4 x 1.5 x 1.8, or say about 10. This would seem ample to account for the observed brittle behaviour.

With such high concentrations of potential energy it becomes easier to understand the frequent occurrence of brittle fractures in close proximity (and often parallel with each other) and the phenomenon of "shatter." It is becoming increasingly apparent that brittle fractures can progress under the impetus of the stored elastic energy in quite small volumes of material in the immediate vicinity of the active fracture front. This view has been expressed in References 2 and 23, and has recently been endorsed by Wells.[37] If this is so, it would seem rewarding to concentrate attention on this small region.

A fundamental question which never seems to have been settled is what determines the stage of flow at which brittle fracture occurs. Roop[20] showed that for similar notched specimens tested at different temperatures, the shape of the load-elongation curves hardly varies (see Figure 7) but the elongation at fracture diminishes with temperature, as shown in Figure 8 (which is plotted from Figure 7). It is important here to distinguish between the load-elongation curves of Figure 7 and the stress-flow curves of earlier illustrations such as Figure 3.

It can be seen from Figure 3 that the true stresses at fracture in plain tensile tests of Project Steel A do not vary greatly over the range of temperatures from +25° C. to -188° C., which suggests that fracture may be determined by a certain critical true stress which increases slightly with decreasing temperature. It is tempting to regard this as a "cohesive strength," but this concept becomes less tenable when it is remembered that the structure and behaviour of the metal are severely altered by the large amounts of plastic flow involved. This probably renders invalid any extrapolations to other forms of loading.

Neither can one conclude that ultimate fracture is dependent only on a certain level of stored elastic energy, since the values of this quantity vary considerably, as seen in Figure 4.

The question of a criterion for rupture must therefore be regarded as unanswered. It is probably true, however, that the attainment of a certain level of locally stored elastic energy is a necessary, though probably not a sufficient, condition for rupture.

THE STEADY STATE

Turning now to the more general mechanics of fracture on specimen scale, for example wide-plate tests, we may review some aspects of the now-conventional approach.

It has been shown elsewhere[21] that the stability of fracturing can be examined by means of an equation of the type:

$$dK/dA = dU/dA - dW/dA - dF/dA \qquad (1)$$

where A is some convenient measure of the current extent of the fracture and U, W, and F are, respectively, the elastic energy released, the work done, and the external energy supplied during the process of fracturing. The fracturing is stable, neutral, or unstable according to whether dK/dA is negative, zero, or positive; i.e., according to whether dU/dA is less than, equal to, or greater than dW/dA + dF/dA. In this context U is the total elastic energy release of the specimen, and not the energy per unit volume as used in earlier equations.

For illustration purposes we may assume a "fixed grip" condition in which no external energy is supplied during fracturing, and we then have the expression:

$$dK/dA = dU/dA - dW/dA \qquad (2)$$

in which the three terms are, respectively, the kinetic energy, the elastic energy, and the work done on the material--all per unit extension of the fracture.

In the absence of direct experimental methods for the measurement of the three terms in Equation 2, one can only speculate on their values, but reasoning has been given elsewhere(2) which suggests a strong probability that after the fracture has progressed a certain distance, a "steady state" is reached, in which the rates, i.e., the three terms in Equation 2, are virtually constant. The arguments in favour of this presumption need not be repeated here, but it is of interest to consider what support for it may be found among recent experimental results.

Rolfe, Lynam, and Hall(6) have shown by careful measurements during the unstable fracturing of 72-in.-wide plates that after the crack had reached a length of about 22 in. the extent, magnitude, and nature of the strain field associated with the advancing tip of the fracture remained essentially unchanged. This is strong, if not conclusive, evidence for the steady state.

There is also evidence that the velocity of such fractures tends to a steady value after an initial period of acceleration and fluctuation. Lazar and Hall(7) have given extensive data on velocity measurements for unstable fractures in 72-in.-wide plates, which, when plotted, yield the effect shown in Figure 9. This shows a distinct tendency for the velocities to settle down to constant terminal values. These fractures were initiated, while the specimens were under known tensile loads, by driving wedges into pre-cut notches. The impact probably accounts for the disturbance in the velocities during the early stages. The initial loads were such as to produce nominal stresses varying from 18 to 20.5 kips/sq in. The initiating impacts and the temperatures of testing varied considerably.

Akita and Ikeda have reported (First Experimental Report, unpublished) similar experiments to those just mentioned, but on specimens 1.2-in. thick and 14-in. wide, and have given velocity measurements for three tests made at the same temperature (-45° C.) and initial nominal stress (10 kg/mm^2), the only variable being the energy of the initiating blow. When replotted, these results appear as in Figure 10, which shows an even more definite tendency for the velocity to attain a constant value. In this diagram, a hypothetical "natural" curve has been added, suggesting the probable velocities in the absence of initial impact. The "natural" curve is plotted from the formula:

$$Vn = Vo\left(1 - \frac{Co}{C}\right). \qquad (3)$$

where Vn is the "natural" velocity, Vo is the terminal velocity (taken at 850 m/sec), Co is the "critical" crack length (taken as 20 m/m) and C is the current crack length. (See Reference 2, Equation 18). It will be noted that the initial disturbance of the measured velocities increases with increasing impact energy, and that the initial slopes of the measured velocity curves also increase markedly with increasing impact energy.

A similar diagram emerges from the data given by Robertson[4] as shown in Figure 11. In this case the initiation was effected by firing a 1/4-in.-diameter bullet through a 3/16-in. hole drilled at the root of a sawcut. It is possible that in this case the velocity fluctuations were to some extent due to reflected elastic waves in the specimen. Taking the velocity of sound in steel at 16,800 ft/sec, it is found that during the time taken by the fracture to traverse the plate (0.9m/sec) each of the two waves from the crack point would traverse the plate about three times, making a total of six traverses. This agrees with the six "waves" in the velocity curve shown in Figure 11.

The behaviour indicated in Figures 9, 10, and 11 suggests that under the conditions considered, i.e., initiation by impact, the velocities may be described in terms of a "natural" curve according to the formula Equation 3 on which is superimposed a damped harmonic oscillation; i.e., by a formula of the type:

$$V = V^{o} \left(1 - \frac{C^{o}}{C}\right) + Be^{-k(C-C^{o})} \sin \frac{2\pi}{r} (C-C^{o}) \qquad (4)$$

in which B, k, and r are constants which may, however, vary according to the initial conditions.

The formula for natural velocities, Equation 3 is of course the result of somewhat oversimplified reasoning[2] and may not indeed correspond with the physical behaviour. The formula is hyperbolic in form, indicating that the terminal velocity is approached asymptotically.

If the concept of a steady state is accepted on the basis of the above evidence, the consequences are interesting. In the first place, it provides support for the theoretical findings of Mott[22] and for the descriptions given in References 2 and 23, which are based on the steady state assumption. It means, however, that the strain energy release rate, dU/dA does not increase indefinitely with crack extension as indicated by the Griffith-Irwin type of approach. It means also that there is probably a limiting time rate at which work, producing fracture, can be performed on a material, and that this value, which can be expressed as a lower limit to dW/dA is an intrinsic property of the material, which may be defined as its "toughness" as suggested in References 2 and 23.

This concept of "toughness" differs from Irwin's G^{c} concept [24,25,26,27] which is a particular value of dU/dA or dW/dA when these two are equal, i.e., at the onset of instability. This particular value, G^{c}, must be expected to vary widely with the initial conditions such as geometry and the nature of the initiating forces, e.g., impact, and is difficult to accept as an intrinsic property of the material.

The steady state concept leads to the type of diagram shown in Figure 12[2] which is a hypothetical plotting of the terms in Equation 2. It is unlikely, however, that such a diagram will be plotted quantitatively until the techniques for measuring the relevant quantities have been much more fully developed.

INITIATION OF BRITTLE FRACTURE

From the diagram, Figure 12, it can be seen that before instability can occur, a certain amount of work, represented by the shaded area in the diagram, must be supplied from external sources. This "initiation energy" is employed in producing the various stages of breakdown at the position where fracturing will occur, i.e., elastic straining, work hardening, shear fracture, micro-fissuring and brittle fracture.

If these actions are localised, for example by an initial geometrical notch, the total initiation energy required will be considerably reduced. If, further, we suppose that unstable fracturing requires a certain intensity of locally stored energy, as suggested earlier, the local strain at which this will be available will be markedly reduced by the triaxiality which will develop as a result of high rates of strain (such as impact if present) and reducing the temperature.

The amount of locally stored energy required must be a function of the condition of the material in the immediate path of the fracture, which we may for brevity call its "brittleness," suggesting that less stored energy is required as the brittleness increases.

The local brittleness may be caused by one or more of several factors, such as the process of breakdown itself, "exhaustion of ductility" due to prior strain,[28,29,30] or precipitation effects such as strain aging and quench aging, often associated with welding.[31]

We see, therefore, that the initiation of unstable brittle fractures at low local strain, and consequently low average stress, is favoured by:

(1) Low temperature

(2) High speed of loading (e.g., impact)

(3) Triaxiality

(4) High local residual stress

(5) Local embrittlement by prior strain

(6) Local embrittlement by metallurgical changes (precipitation)

(7) Pre-existing microcracks.

The first four of these operate by increasing the locally stored elastic energy at low strains, and the remaining three by reducing the demand for locally stored energy. The last four are frequently associated with welding, which probably accounts for the relative ease with which low-load brittle fractures can be initiated in the vicinity of welding.

At this point it is of interest to observe that the conditions at the point of initiation control the overall load, and consequently the nominal stress at which instability sets in.

Thus, in the absence of artificial initiation, e.g., by blows, wedges, bullets and the like, the nominal stress at which instability will occur must be greatest in specimens presenting the greatest resistance in the initial stages.

This critical load and nominal stress is that which is required to cause the small volumes of material at the point of initiation to go through all the necessary stages of breakdown, and to store the necessary elastic energy for subsequent unstable rupture. It is found that in virgin material, however sharply notched, this load is of the order of that which causes general yielding of the specimen; but when impact, low temperatures, or local embrittlement intervene, the critical load may be very much less.[32]

Many service fractures have occurred at nominal stresses well below those associated with general yielding, and indeed general yielding seems to be exceptional in service brittle fractures.

UNANSWERED QUESTIONS

From the general discussion it is apparent that a great deal still remains to be learned before we can construct a theory from which confident predictions can be made on the basis of known and controllable data.

In spite of the extremely valuable and extensive work which has been done, typified for example by the report of the Swampscott Conference,[12] it is clear that a great deal of further work will be required to provide a better understanding of the processes of breakdown of metals. Nevertheless, if the knowledge already available could be more thoroughly assimilated by a greater number of workers in the field, it is probable that considerable progress could be made.

The root of the problem seems to lie in the fact that a metal is capable of many different responses to loading; and the crucial question is to determine what it is that controls the choice of these alternative responses.

In particular, the question already mentioned as to what controls the stage of flow at which fracture ensues, has not yet been answered.

Much remains to be done on the effects of speeds of loading and deformation, on the lines of the works cited and of the recent contribution by Krafft and Sullivan.[33] This latter, incidentally, emphasises the subjective nature of deformation rates and speeds of loading.

There is as yet no measurable property of a material whereby its fracture behaviour in a notched specimen or in a structure can be confidently predicted. This is a major challenge to all workers in this field.

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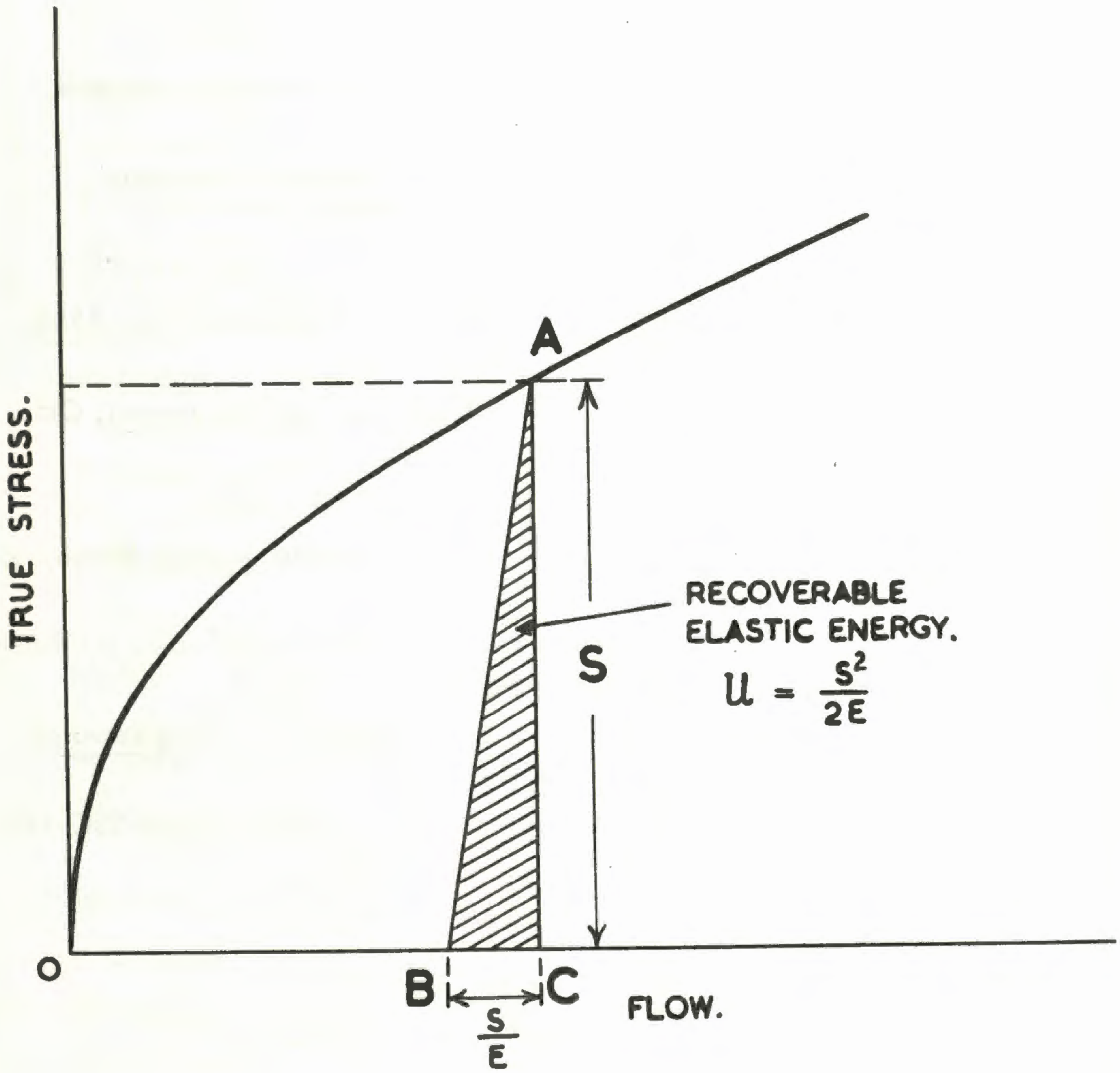

Figure 1 Idealized stress-flow relationship.

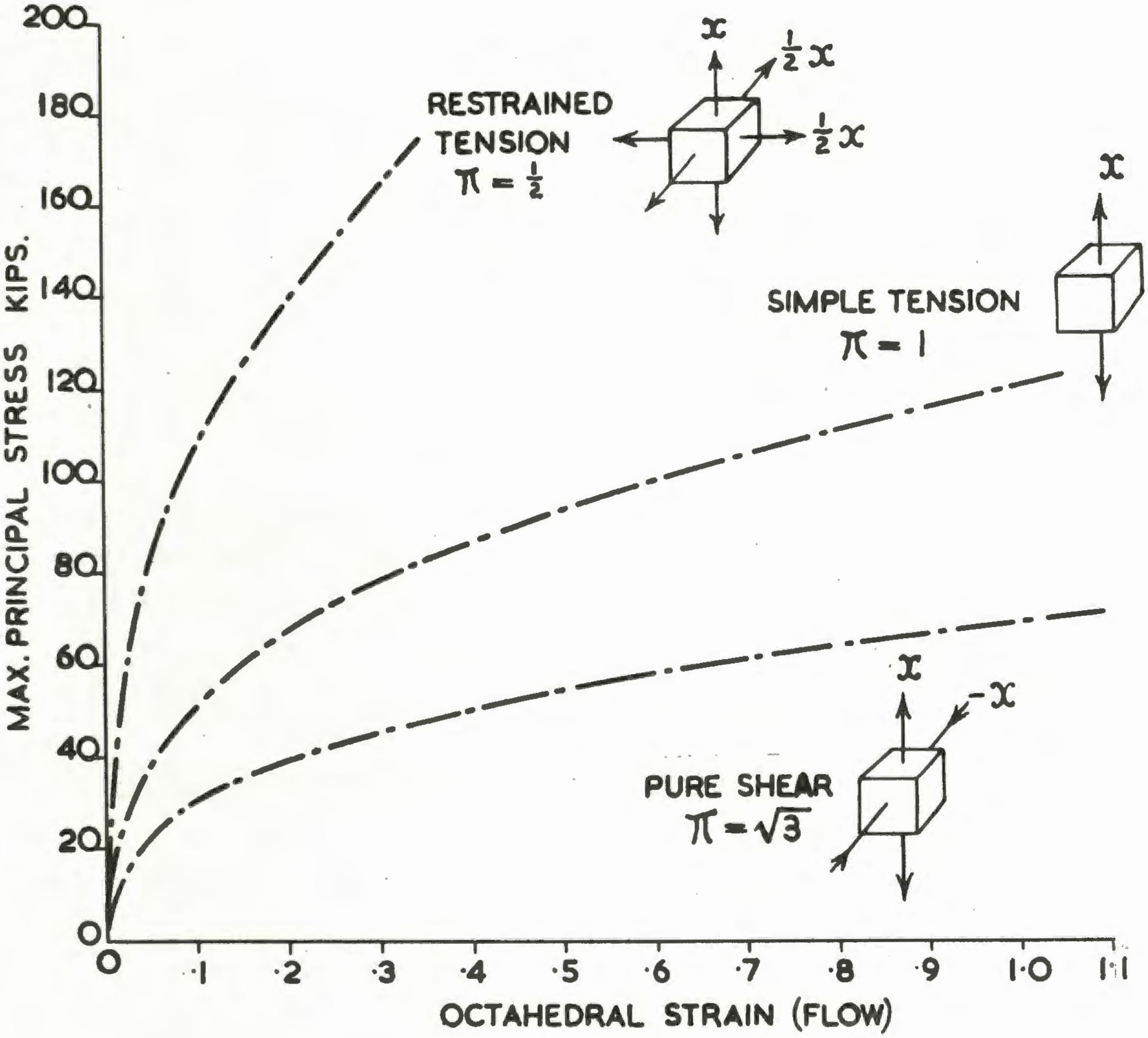

Figure 2 Effect of triaxiality on stress-flow relationship.

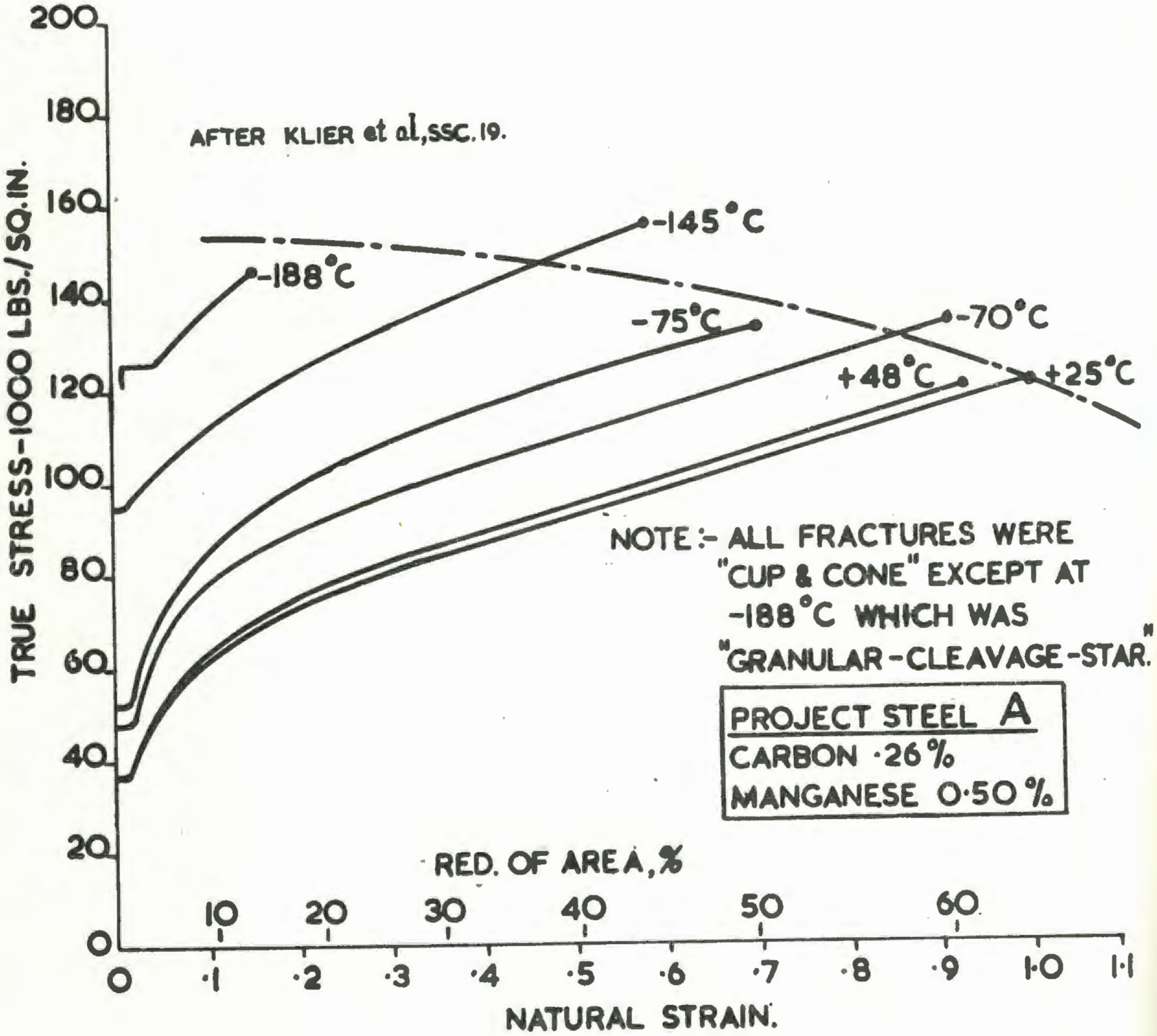

Figure 3 Effect of temperature on the stress-flow relationship.

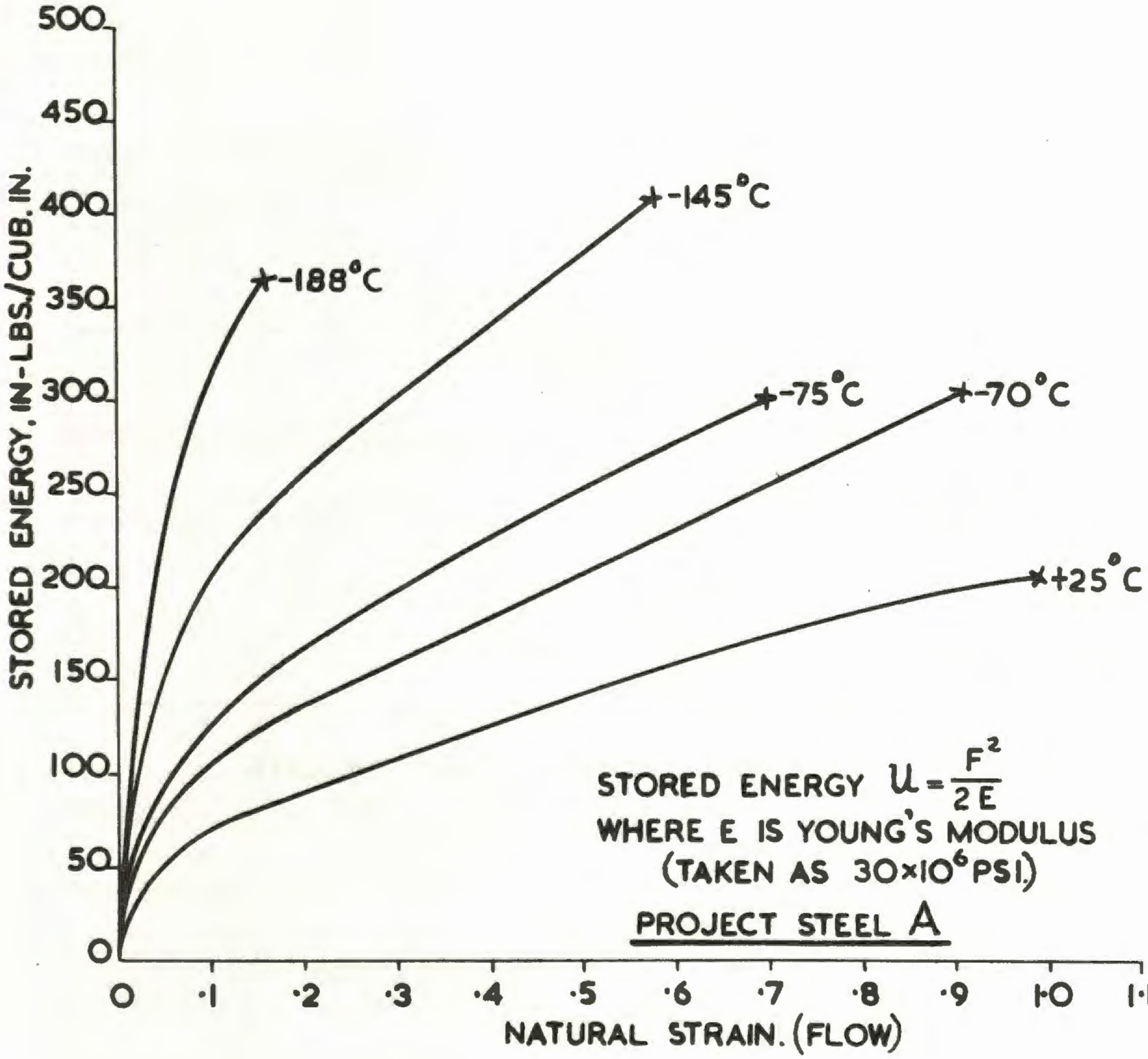

Figure 4 Stored energy versus flow for various temperatures.

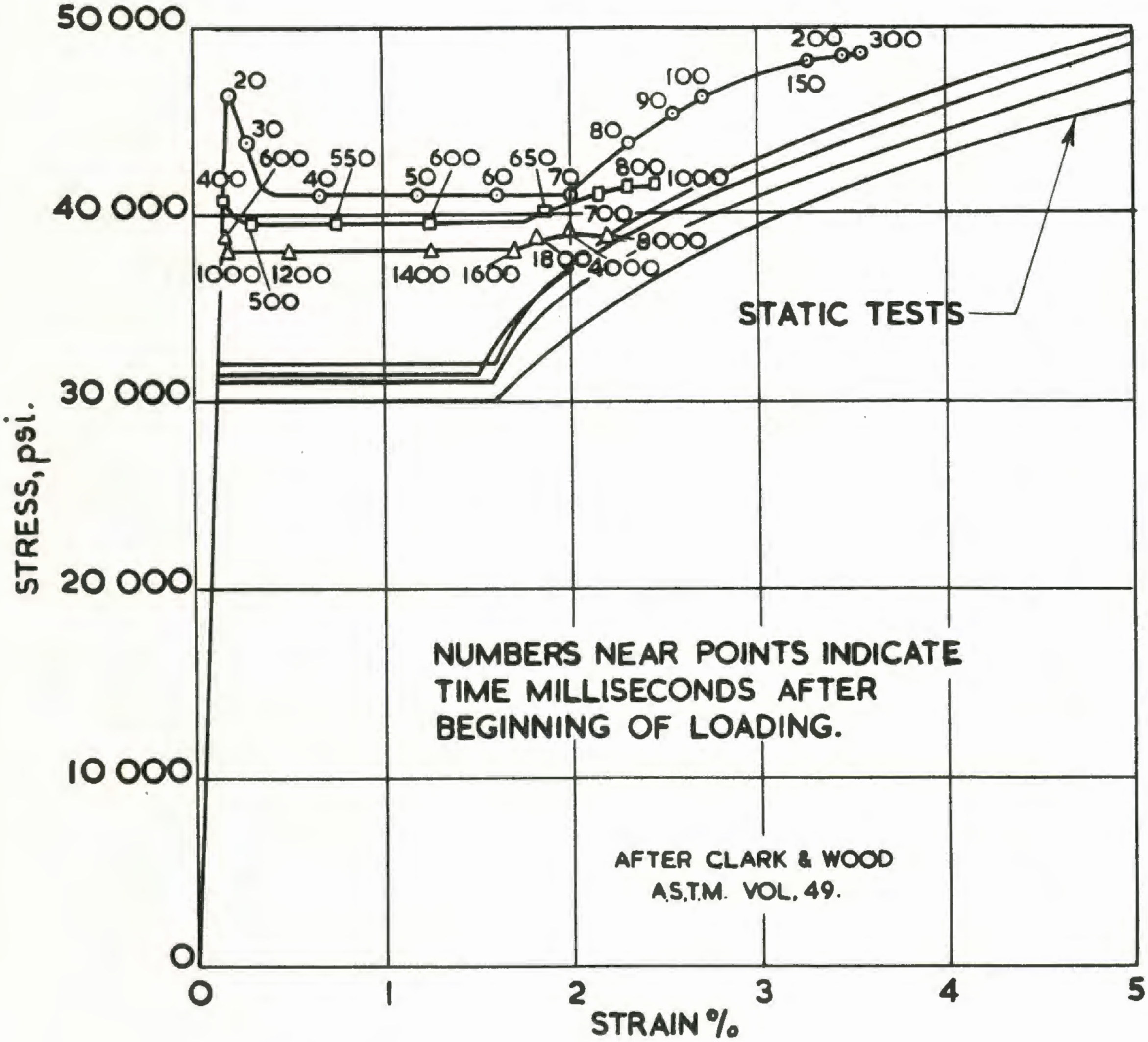

Figure 5 Rapid load tests on 0.19 per cent carbon annealed steel.

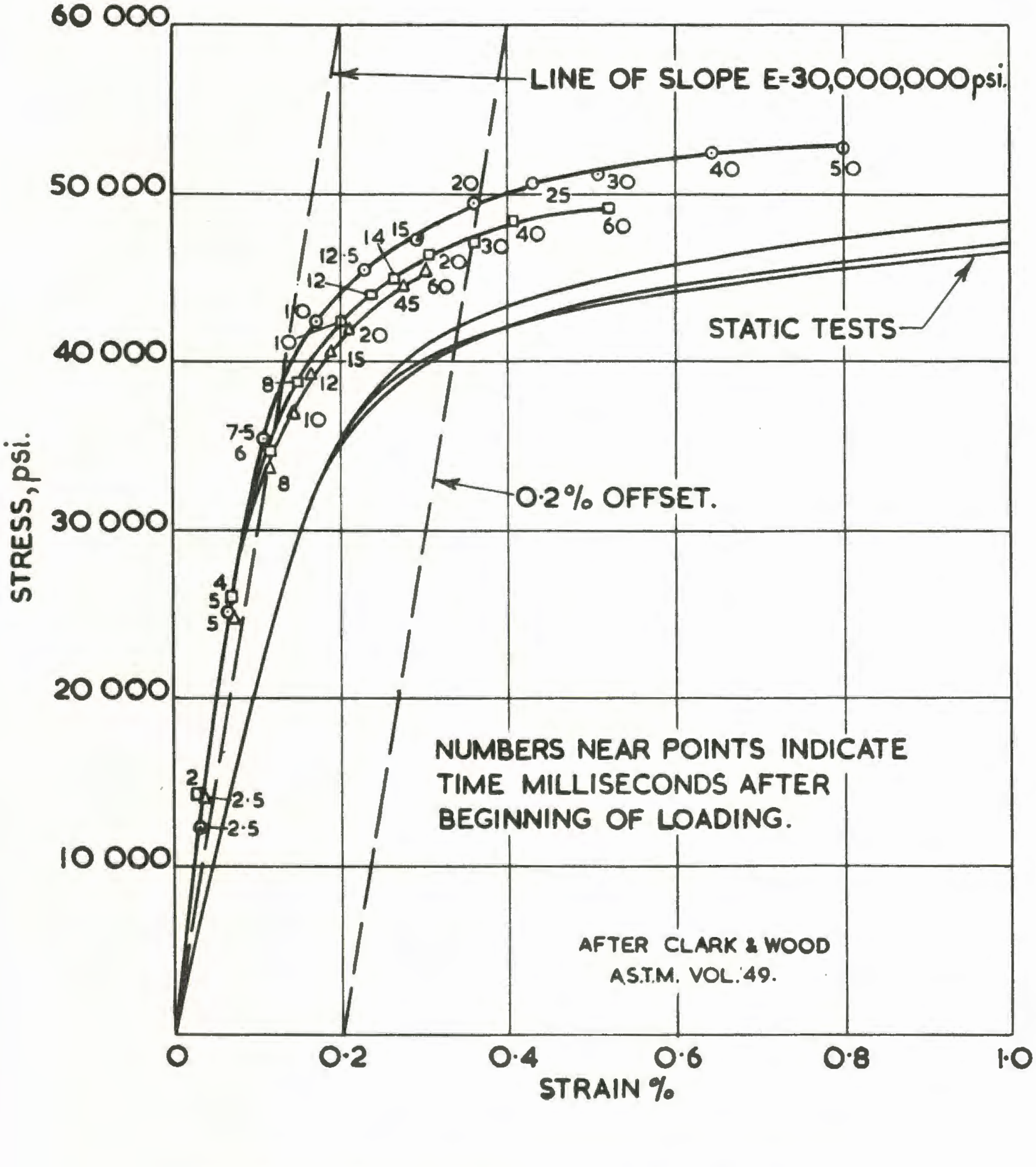

Figure 6 Rapid load tests on type 302 stainless steel.

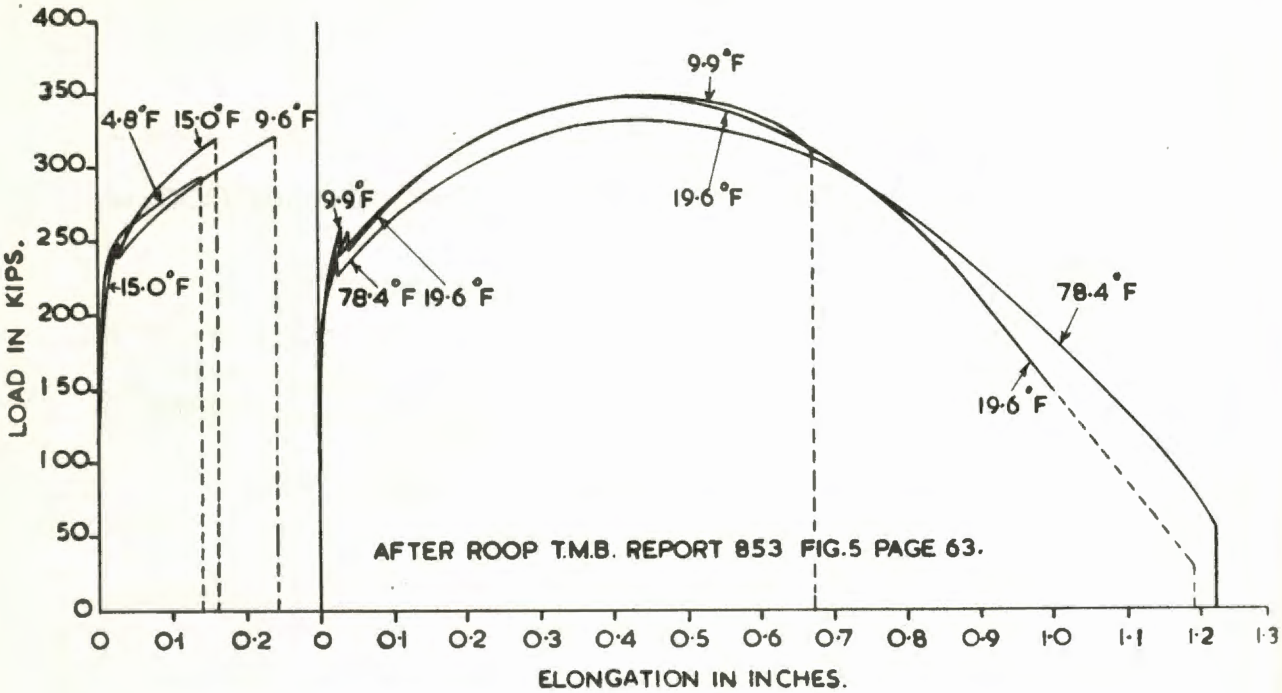

Figure 7 Load elongation curves for selected specimens in subgroup 26.

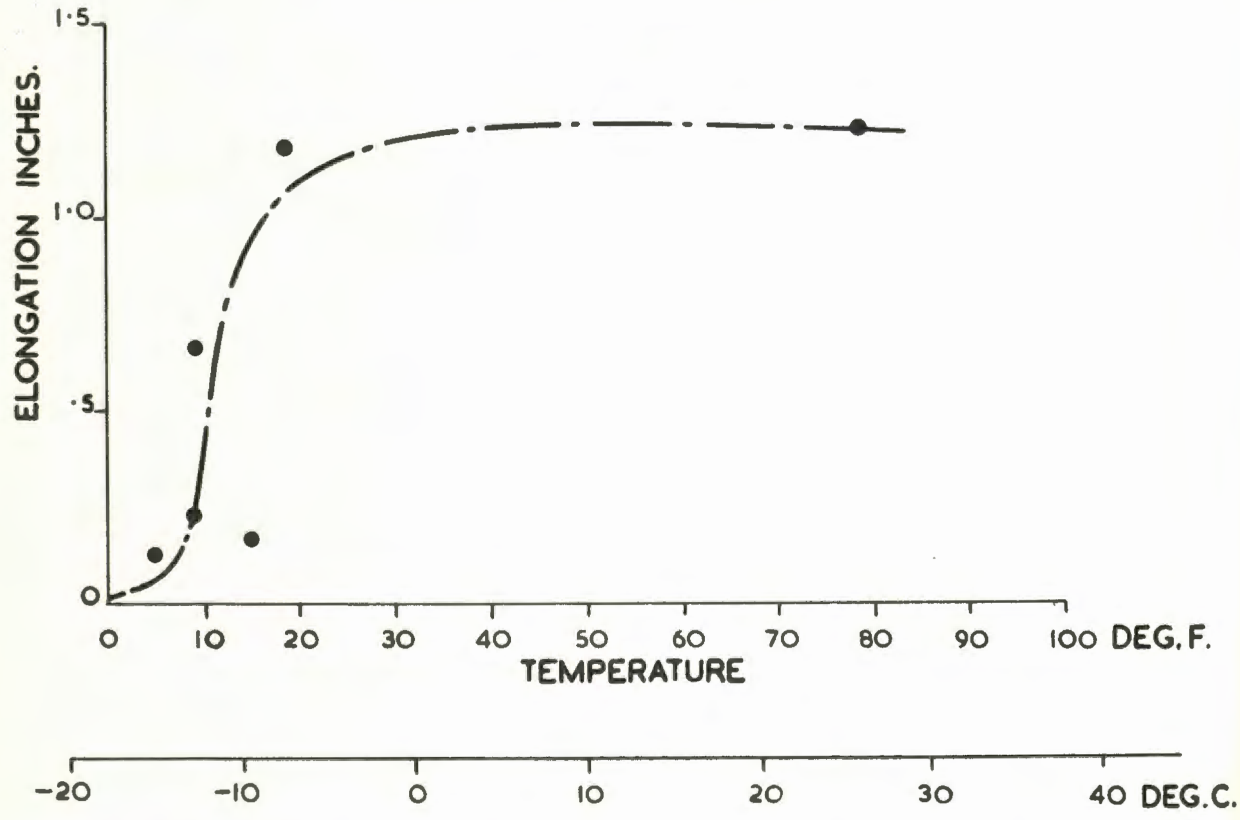

Figure 8 Effect of temperature on elongation at fracture in notched bar tests.

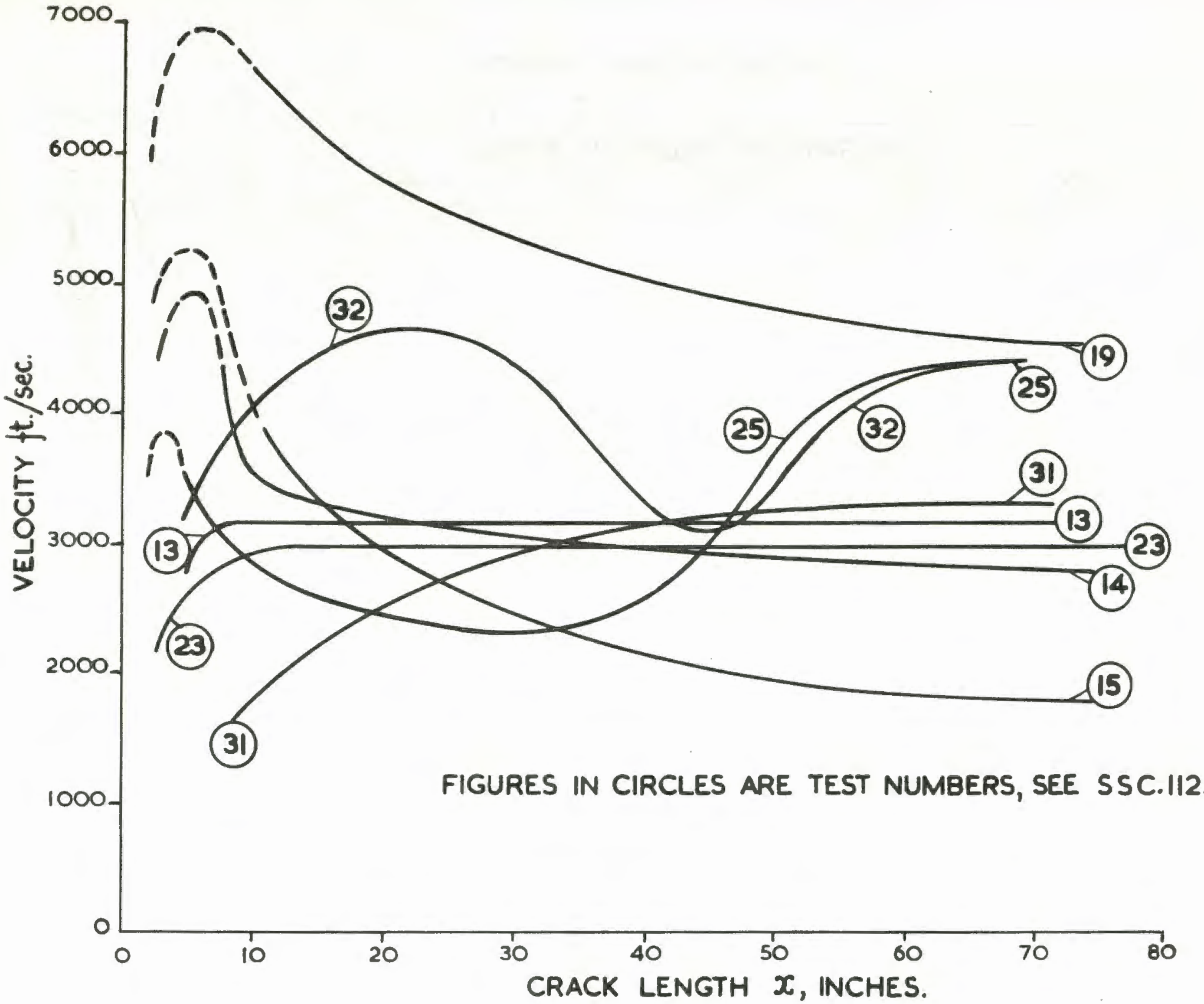

Figure 9 Velocity versus crack length--72 in. wide plate tests.

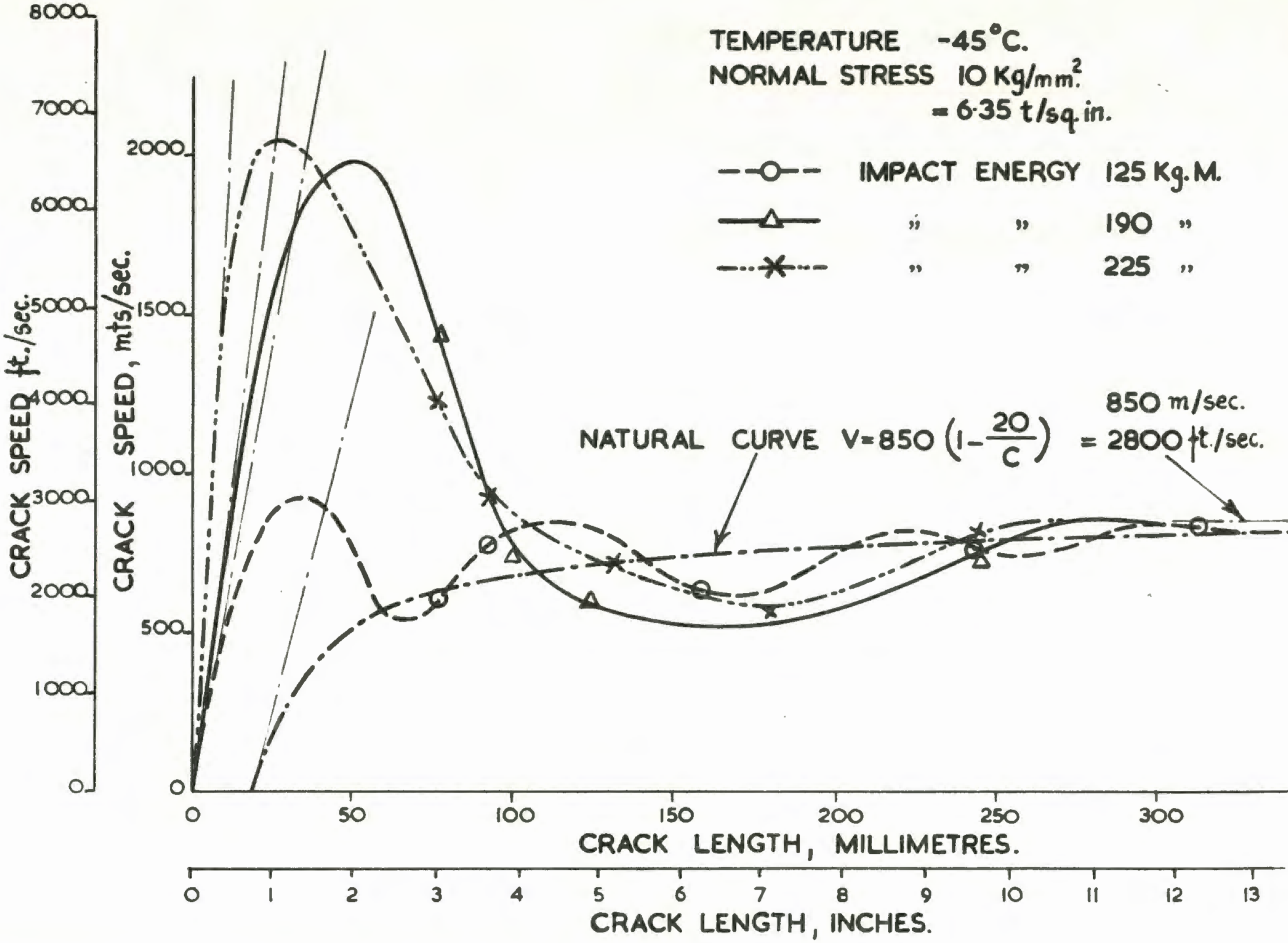

Figure 10 Crack speed versus crack length—14 in. wide plate tests.

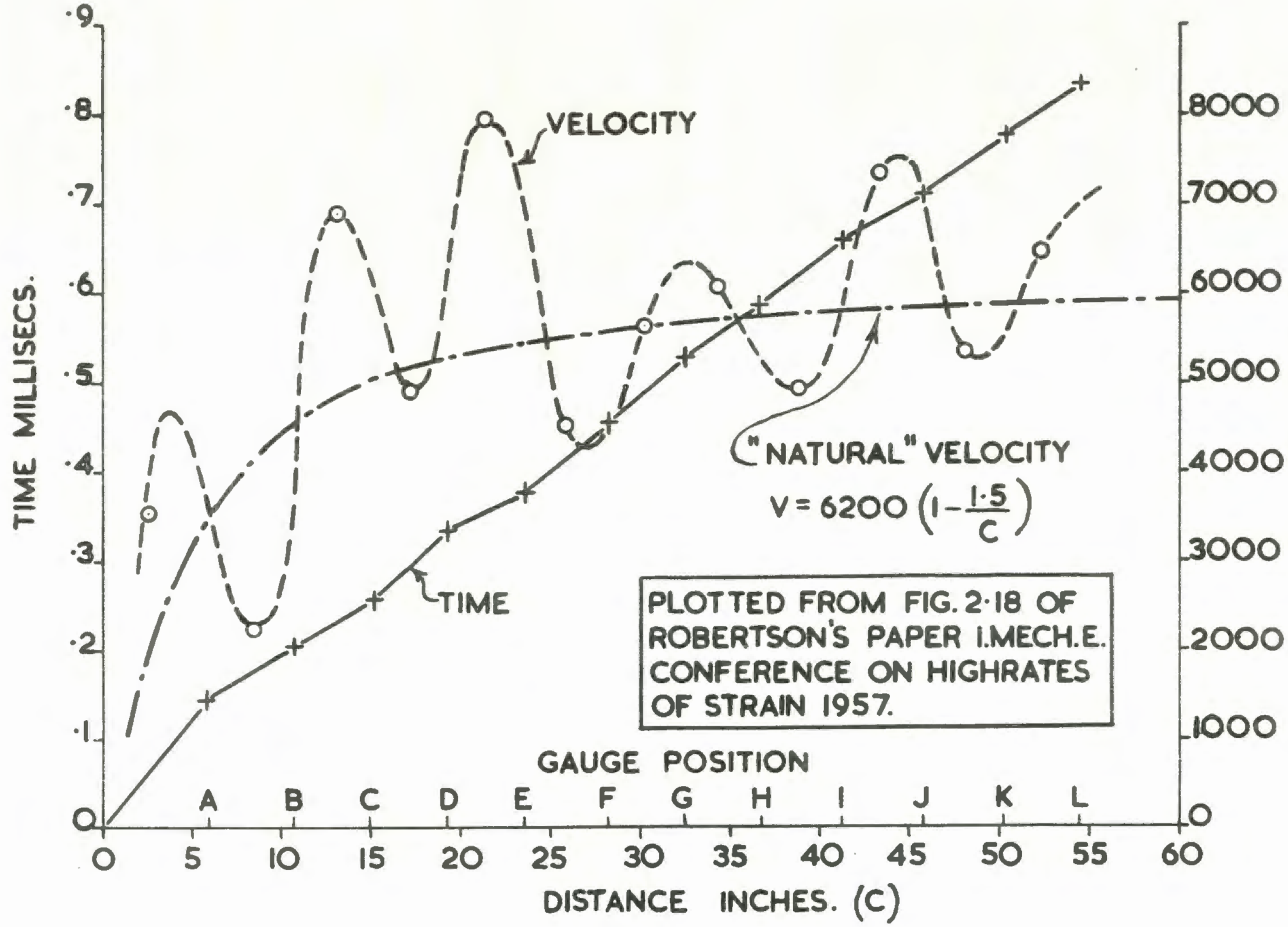

Figure 11 Velocity versus crack length.

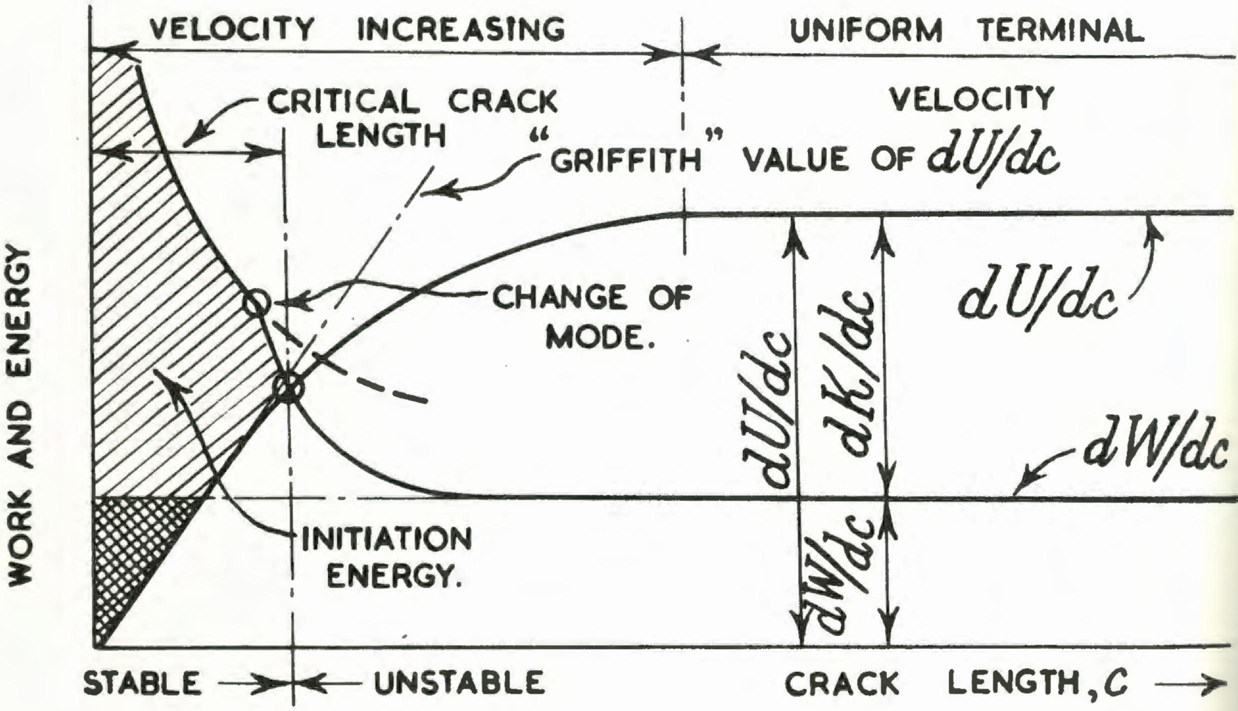

Figure 12 Illustration of work, energy and crack length concepts.

INFLUENCE OF RESIDUAL STRESSES ON BRITTLE FRACTURE

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INTRODUCTION

I received your kind invitation at the same moment that my belief in residual stresses had come to a rather critical stage. A short time ago I took it for granted that residual stresses were really something dangerous; now I have started to doubt if they are really as dangerous as generally has been accepted.

INITIATION AND PROPAGATION

At present, we are accustomed to dividing the phenomenon of brittle fracture in two distinct phases: initiation and propagation. Propagation of a brittle crack is in fact a process of continuous dynamic initiation and it will always remain difficult to draw a sharp dividing line between these two phases.

However, on the one hand, the static initiation of a brittle failure in homogeneous materials is always more difficult to explain than the dynamic propagation. If conditions for initiation are fulfilled over a large part of the specimen, then propagation of the crack will certainly take place with ease. The contrary, however, is not true: a crack will not necessarily initiate even if the material is in the right condition for propagation of the brittle fracture. On the other hand, we know also that there may exist profound differences in properties of the metal at the loci where initiation can take place and along the path where the crack is expected to propagate. In general, the origin of a fracture can be traced back to some sort of a static crack. The metal at the tip of this crack may possess mechanical and structural properties which are different from those of the virgin metal. For both reasons it is thus necessary to separate, in the study of brittle fracture, the phenomenon of initiation and the phenomenon of propagation.

THE INITIATION OF A BRITTLE FRACTURE

It may be worthwhile to define clearly what we mean by initiation because, in some instances, it has led to confusion. A fracture will initiate at some sort of discontinuity in the specimen, a discontinuity of either physical, chemical, physio-chemical, or some other nature. In practice, fractures usually initiate at a physical discontinuity, e.g., weld defects, gas pockets, notches, sharp transitions, pre-existing cracks, etc.; the shape of this critical discontinuity will play a major part in the phenomenon of initiation. Hence it is logical to accept the most severe conditions: that is, the existence of a static crack in the specimen. Such a crack might have been originated by different processes: fatigue, creep, corrosion, etc. Obviously, these cracks may possess different properties because the metal adjacent to the tip of the crack may have been subjected to more or less severe structural damage. In fact we have a heterogeneous material with the initiation point in the deteriorated part. And we can already foresee that the type of damage may have a more pronounced effect in the phenomenon of initiation than the presence or absence of residual stresses.

Quite frequently such cracks are found to have been originated by a fatigue process. Therefore it seems logical to study brittle fracture initiation on specimens precracked by repeated loading. We believe that, in doing so, one imposes more severe conditions than by machining a less damaging notch. Furthermore we suppose that the fatigue crack extends throughout the full thickness of the plate but eventually may have different lengths.

When we load such a specimen in tension, it will rupture in either a brittle or ductile manner, depending on the test temperature. We call the transition temperature the highest temperature at which brittle fracture still occurs. Several tests have indicated that this transition temperature for a given plate thickness is independent of the initial length of the fatigue crack. All fractures occurring below this temperature will be called brittle, while fractures occurring above this temperature will be called ductile. This definition of brittle and ductile fracture is based on fracture appearance and has no correlation with fracture strength.

This is important because some researchers link the brittleness of the material with the fracture strength.

Since we are only interested in brittle fractures we assume that all our test specimens are fractured below this transition temperature. The strength at fracture is called the brittle strength.

The steel user has to accept the fact that below the transition temperature, a pre-cracked specimen will fail in a brittle manner. But the crux of the matter is that in some instances structures have failed at extremely low external loads. The subject of this talk will be to explore if residual stresses have something to do with these low stress failures.

STRAIN RAISERS

We all know that we have a stress concentration at a discontinuity and we ascribe to different kinds of notches a stress concentration factor (S.C.F.). If we load a precracked specimen in tension, then the stress at the tip of the notch reaches yield-point values very rapidly. It is only of academic interest to know if the material at the root of the notch will behave elastically (which is possible, due to the triaxiality) or plastically during further loading. The fact is that total yielding only takes place when the average stress on the cross section reaches the yield-point value of the metal. The stresses at the tip of the notch arise as a consequence of the compatibility conditions, which are independant of the stress-strain relationship. Hence strains at the tip of the notch, even in the plastic flat portion of the tensile curve, are several times larger than the strains existing in the rest of the specimen. For this reason it is more logical to talk about a strain concentration factor than a stress concentration factor. At the tip of the notch very large strains are generated by a relatively small external load. The presence of residual stresses will contribute very little to those strains.

It is of great interest to know the critical strain at which brittle fracture may initiate. So far, little work has been done to investigate that aspect of fracture initiation. Dechaene and Vinckier,[(1)*] using the Moiré technique to visualize plastic deformation, gave plastic iso-strain curves of notched tensile bars. Dechaene made a complete study of the plastic deformation of cracked specimens. Briefly summarized, the Moiré technique works as follows: one draws a set of regularly spaced lines on the test specimen and a similar set of lines on a glass plate. The specimen is then deformed into the plastic region. When one now superimposes the undeformed grid of the glass plate upon the deformed grid of the specimen, then some pattern of interference fringes are created by the mismatch between the two grids (Figures 1, 2, 3). It is easy to see that one can calculate from this interference pattern the amount of deformation. Indeed, the fringes form the loci of equal displacements.

Dechaene performed his tests on a 5 1/2-in.-wide mild steel specimen containing a 1/2-in.-long fatigue crack in its center. Full yield occurred at 39,200 psi. (27.6 kg/mm^2) and the extension at the apex of the notch amounted then to 16 per cent measured on a 0.006-in. base. During further plastic deformation, the stress fell to 38,400 psi. (27.0 kg/mm^2) and with the Moiré technique it was easy to follow and measure strains in Lüders' bands ($\pm$ 2 per cent) while the strain at the tip of the crack increased at the same time to $\pm$ 75 per cent. It is likely that even larger strains could be detected if one used a shorter measuring base for determining them. (Figures 4, 5, 6, 7) The crystals at the tip of the notch must be subjected to very large strains even before full yielding takes place, that is, for average stresses well below the yield point. If the material at the tip of the notch is no longer capable of following such strains, then fracture will initiate.

EMBRITTLEMENT

It is known that several treatments can drastically decrease the deformability of mild steel. The ductility is diminished in unnotched mild-steel

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* Superscript numbers in parentheses refer to references listed in the Bibliography.

specimens, for which the natural strain at fracture is of the order of magnitude of 100 per cent at room temperature, by:

(1) Temperature: Necking disappears at temperatures of the order of -120° C. and the elongation at fracture is only of the order of 30 to 40 per cent. At $+200^\circ$ to $+300^\circ$ C. the elongation at fracture can drop to 70 per cent (Figure 8).[(2)]

(2) Prestraining: If a specimen is prestrained in tension at room temperature to 40 per cent, then the remaining ductility will be of the order of 60 per cent, the total ductility being reduced by its prestraining value.[(3)]

(3) Prestraining and temperature: If a specimen is prestrained to about 60 per cent at 200° C., then the natural strain at fracture in room-temperature tests may drop to 5 to 10 per cent. It is generally assumed that this drastic decrease in ductility results from longer time exposure ($\simeq$ 2 hours) to such temperature. But it has been shown that very short heating times, of the order of seconds, is sufficient to decrease drastically the room temperature ductility.[(4)]

An even larger drop in ductility may be expected in notched tensile specimens, where the ductility is already decreased by triaxiality.

True strain measured on cylindrical specimens decreases to $\pm$ 8 per cent at temperatures of 200° to 300° C., whereas at room temperature values of 30 per cent are measured.[(2)] (Figures 8, 9)

Some tests carried out by Japanese researchers[(5)] indicate that the embrittlement is more severe when straining is carried out at temperatures of the order of 200° to 300° C., as compared to specimens which are strained at room temperature and then brought to ageing temperatures. There are several reasons to believe that the mechanism of embrittlement due to straining at high temperatures is not governed by a standard ageing process. This embrittlement, as already mentioned above, seems to occur exceedingly fast. It is interesting to note that the metal does not seem to embrittle when it cools from temperatures above 400° C., and goes through the ageing range. We may be happy that things are this way, otherwise every weld with the slightest defect should hardly be capable of deforming more than 10 per cent.

It should be noted that heat treatments at temperatures above 400° C. restore very quickly the ductility of mild steel. Such a thermal treatment is thus very favorable, and so even more should heat treatments at 600° C., as generally applied for stress-relieving treatments. Is the increase in ductility obtained by our standard stress-relieving treatment now caused by a metallurgical restoration of ductility or by merely a decrease of our residual stresses? I ~~don't feel~~ am inclined to believe that this beneficial effect is due to a purely metallurgical restoration. If so, then we would gain considerably in the expense of stress-relieving heat treatments, by just lowering the temperatures from 600° C. as generally prescribed, to 450° C. Moreover, soaking times could also be drastically reduced from hours to minutes.

STRAIN RAISERS VERSUS EMBRITTLEMENT

Let us go back now to the strains arising in a cracked plate. We have seen that at the tip of the notch we may expect strains of the order of 75 per cent at the moment of full yielding of the specimen. This strain is comparable to the natural strain at fracture of embrittled steel. This may be an indication that under certain circumstances a precracked specimen may fail at an average stress lower than the yield stress, even in complete absence of residual stresses.

Residual stresses should influence the initiation of brittle failures only in so far as they contribute to the elastic or plastic prestraining. Their role should then be of only minor importance in virgin material; the most important aspect of the problem is the prior embrittlement at existing notches or cracks.

EXPERIMENTS

In order to investigate the above considerations, we should set up our experiments in the following way: either (1) try to prove that it is possible, in the absence of residual stresses, to initiate a brittle fracture with low external loads; or (2) try to prove that the presence of residual stresses has no or only a minor influence on the initiation of a brittle fracture.

The first series of tests will be difficult to carry out. How could we prestrain a cracked specimen without introducing residual stresses during the loading and unloading cycle? It may be that one could eliminate them by making a permanent indentation on the lateral surfaces close to the notch or crack as was done by Professor Mylonas of Brown University.[6] Without going into detail on the techniques of these tests we quote his conclusion: "It may be stated that in the tests the removal of the existing residual tensile stresses did not alter appreciably the type of fracture (low stress fracture), and hence in these particular tests the residual stresses do not appear to have been a significant factor in the fracture."

Tests have been carried out in Ghent to prove that residual stresses alone have no influence on the brittle strength.[7]

The purpose of the first series of tests was to ascertain if the residual stresses introduced in a cracked piece, without heating the cracked zone, may decrease the brittle strength. This verification is very important; in fact, due to the tests of Wells and Kihara, the deleterious effect of residual stresses is generally accepted.[8,9,10] However, both of these investigators have obtained cases of low brittle strength in the laboratory by initiating the brittle fracture in a zone which is subjected to the influence of welding, i.e., not only the effect of residual stresses, but also the effect of heating and of plastic deformation. We have tried to produce a test where the residual stresses are introduced into the cracked portion of the specimens without heating this zone. These tests have been made as follows: in plates of 70 x 14 mm (2.76 in. x 0.55 in.) section, a hole of 5-mm (0.199 in.) diameter has been drilled in the center; these test pieces have been loaded in fatigue until beginning of cracking, the width of the cracked part being about

10 mm (0.394 in.) (Figures 10,11). These test pieces have then been welded in rigid frames where the columns had a section of about 2.2 times the section of the cracked piece. Before welding the web, strain gages had been placed on the six accessible sides of the columns. These gages make it possible to determine exactly the value of the compression stresses in the columns during the welding of the web. Knowing this stress, the compression load in the columns is known, which, in the absence of all external forces, must be equal to the tensile load in the web. One only has to divide this force by the section of the web in order to know the average stress in the web, i.e., the residual stress that acts upon the cracked piece. The distribution of the residual stresses in the frame is indicated in Figure 12. This distribution is comparable to that of the longitudinal stresses of a butt weld shown in Figure 13. If, moreover, one considers the fatigue crack present in the web, our test piece fairly well approaches the one used by Kennedy and Wells;[8,10] the only essential difference is that in our test piece the cracked section has not been heated.

By measuring the strains in the columns, when pulling on the whole specimen, it is possible to know the part of the load carried by the columns and the rest of the load carried by the web. These measurements enable us to determine, for each external load, the stresses in the columns and the web.

The stresses in the web and in the columns may be presented in a theoretical diagram where the stresses in the columns and in the web are plotted against the average external stress $\sigma = P/A$ (Figure 14). The residual stresses in the web and the columns are respectively represented by OA and OA'. As long as the web is elastic, the increase of the stresses in the web and in the columns is the same, which is indicated by the lines AB and A'B'. If the web becomes plastic, its stress remains constant and equal to the elastic limit σy, which is indicated by the horizontal line BE. The column has to take all of the increase in external load which is indicated by the straight line B'E; the slope of B'E is given by the ratio A/Ac, A being the total section of the test piece and Ac the section of the columns. One can easily verify that the columns will reach the yield point when the external stress reaches the yield point. The tension tests were performed at -20° C., i.e., below the transition temperature of the steel. Examination of the experimental diagrams (Figures 15, 16) reveals their similarity with the theoretical diagram. The results are summarized in Table 1.

TABLE I - B STEEL

Specimen: frame type (Figures 10 and 11)

Chronological treatments:
(1) Drilling of a hole 5-mm diameter in the center of a plane specimen
(2) Fatigue test on the specimen till cracking
(3) Welding of the specimen in the frame (residual stresses are set up)
(4) Tensile test a -20° C.

Test piece No.	21 (Fig.15)	22	23	39
Section of the web - mm^2				
before cracking	70x14=980	70x14=980	70x14=980	70x14=980
after cracking	58.5x14= 819	58x14= 812	57.3x14= 802	58.5x13.3= 778
Section of the columns - mm^2	1966	1854	1915	1931
Total section - mm^2	2785	2666	2717	2709
Residual stresses - kg/mm^2				
in the web	+29.4	+27.8	+27.8	+25.9
in the columns	-12.3	-12.2	-11.4	-10.5
Stresses when the web becomes plastic at -20° C. kg/mm^2				
in the web	+34.0	+31.0	+31.8	+28.6
in the columns	- 9.4	-11.0	- 8.8	- 8.4
external	+ 3.2	+ 2.3	+ 3.2	+ 2.2
Stresses when the columns become plastic at -20° C. kg/mm^2				
in the web	+32.0	+38.5	+37.0	+31.9
in the columns	+20.5	+22.0	+20.5	+21.3
external	+23.3	+26.0	+25.5	+24.4
Fracture stress - kg/mm^2				
external by -20° C.	+25.0	+31.3	+27.6	+25.5

From these tests one can draw the following conclusion: Notwithstanding the presence of high residual stresses, the brittle strength of the whole test piece reaches values almost identical to those obtained on test pieces without residual stresses. This conclusion can easily be explained by the diagrams. If indeed the web, although being loaded to stresses of the order of the yield point, possesses a deformation capacity sufficient to follow the elastic deformations of the columns, the whole test piece will resist an average stress equal to the yield point. But as these elastic deformations are very small, it is likely that a cracked piece would possess sufficient ductility to be able to follow this elastic deformation. In our tests, this ductility was amply present. It seems logical to conclude from these tests that the brittle fracture does not occur because the average stress reaches a certain critical value, but because a critical deformation has been reached. Thus the criterion of fracture does not seem to be a function of the stresses.

The second series of tests were conducted on specimens cracked by fatigue in which residual stresses were set up by subsequent heating; the specimens were then loaded until fracture occurred. This succession of operations corresponds at least qualitatively to phenomena which occur during welding. In principle these tests should reproduce the interesting cases of brittle fracture at low strength.

Two series of tests have been made. In the first series the test pieces of the frame type have been completely machined without welding (Figure 16). In the web a hole of 5 mm was drilled and the entire test piece, web and column, were cycled in fatigue with a maximum stress of 35,000 psi. (25 kg/mm^2). The cycling was stopped when the fatigue crack initiated in the hole had reached a length of about 10 mm (0.394 in.). Residual stresses were then introduced by heating the web with an oxyacetylene torch while the columns were kept at room temperature.

In the other series of tests, the web without its columns was cycled in fatigue after drilling a hole for the initiation of the crack. When the crack had reached a length of about 10 mm (0.394 in.), the cycling was stopped and the complete test piece was heated in a furnace at a temperature of 250° C. for one hour. After this heat treatment the test piece was welded in its frame, setting up important residual tensile stresses in the web.

The essential difference between these two series are that in the first series the heating (and eventual embrittling) and the introduction of residual stresses were made together, whereas in the second the heating was carried out on the elements free of stresses.

The results of these tests are summarized in the following tables. We examine first the results of the first series in Table II.

TABLE II - B STEEL

Specimen: frame type (Figure 16) without welds.

Chronological treatments:
(1) Drilling of a hole in the web of a frame type specimen without welds.
(2) Fatigue test on the frame till cracking occurs in the web
(3) Heating the web with an oxyocetylene torch (residual stresses are set up)
(4) Tensile test at -20° C.

Test piece No.	13	14	15
Section of the web			
before cracking - mm^2	730	711	735
after cracking - mm^2	577	547	557
Section of the columns - mm^2	1219	1195	1232
Total section - mm^2	1796	1742	1789
Residual stresses - kg/mm^2			
in the web	+30.0	+29.8	+28.3
in the columns	-14.2	-13.7	-12.8
Temperature of tensile test - ° C.	-20	-20	-20
Stresses at the moment of fracture of the web			
in the web - kg/mm^2	30+2.5=36.5	29.8+8=37.9	28.3+10=38.3
in the columns - kg/mm^2	-14.2+3.5= -10.7	-13.7+8.8= -4.9	-12.8+3.6= -9.2
external - kg/mm^2	4.6	8.6	5.6
Strain of the columns at the moment of fracture	175.10^{-6}	442.10^{-6}	180.10^{-6}
Aspect	brittle	brittle	brittle

The most important fact is certainly the fracturing of the webs of the three test pieces at exceptionally low strengths, respectively 6,500; 12,000; and 7,900 psi. (4.6; 8.6 and 5.6 kg/mm^2). This result does correspond to cases of certain welded structures which have collapsed under very low external loads. For somebody who ignores the state of strain existing in the web before application of the external loading, these fractures are unexplicable, as indeed are the fractures of some collapsed welded structures.

One must thus conclude from these tests that the technique of heating the clamped web with an oxyacetylene torch has embrittled the cracked section of the web.

It appears from the results of Table III that of the nine test pieces, three were broken under low external load; the stresses which caused the fracture of the webs being respectively 4,650, 6,050, and 24,000 psi. (3.3, 4.3 and 17.0 kg/mm^2).

The results of the first series of tests, where all test pieces broke under very low stresses, seem to indicate that the embrittlement is due to a pre-straining effect at a critical temperature when a load is applied. By the combined action of shrinkage and cooling, the prestraining takes place at all temperatures and thus also at temperatures where embrittlement occurs.

The results of these two series of tests are in close agreement with results obtained by Terazawa and co-authors.[3,5] In their paper these authors claim that embrittlement caused by straining at high temperatures (200° to 300° C.) is twice as large as embrittlement caused by straining at room temperature followed by a 250° C. heat treatment. Their statement may explain why all specimens of the first series broke at low stresses, against only three out of nine specimens of the second series, which were strained at room temperature and heated at 250° C.

CONCLUSION

The initiation of low stresses fractures can only be explained by taking into consideration the inhomogeneity of the steel. It must be considered as

TABLE III - B STEEL

Specimen: frame type (Figures 10 and 11)

Chronological treatments:

(1) Drilling of a hole in a plane specimen
(2) Fatigue test on the specimen till cracking
(3) Heating of the specimen in a furnace at 250° C.
(4) Welding of the specimen in the frame (residual stresses are set up)
(5) Tensile test at -20° C.

Test pieces n°	25	26	24	48	47	46	51	49	50
Section of the web									
before cracking - mm^2	980	980	980	980	980	980	980	980	980
after cracking - mm^2	825	821	754	783	-	782	-	-	-
Section of the columns mm^2	1919	1920	1966	1915	1854	1966	1920	1931	1918
Total section - mm^2	2744	2741	2720	2680	2661	2748	-	-	-
Residual stresses - kg/mm^2									
in the web	29.5	26.1	29.7	26.8	27.8	31.3	28.7	-	29.2
in the columns	-12.7	-11.1	-11.4	-10.9	-12.1	-12.4	-11.3	-11.8	-11.8
Stresses when the web becomes plastic - kg/mm^2									
in the web	31.0	30.21	-	29.7	29.02	34.5	31.8	-	30.5
in the columns	-12	-8.23	-	-7.5	-6.2	-9.4	-9.5	-	-11.5
external	1.0	3.28	-	3.3	4.5	3.3	2.2	-	1.0
Stresses when the columns become plastic - kg/mm^2									
in the web	35.8	30.0	-	30.5	33.7	38.7	-	-	32.5
in the columns	20.6	16.0	-	22.0	20.9	22.8	-	-	21.3
external	25.2	20.0	-	24.5	24.8	27.3	-	-	23.5
Stresses at the very moment of fracture of web									
web	-	-	34.5	-	-	-	30.8	-	-
columns	-	-	-8.7	-	-	-	-7.5	-	-
external	26.4	26.5	3.3	26.5	28.2	28.5	4.3	17	26.0

proven that some treatments are able to damage very seriously the ductility of steel at the bottom of a crack. This embrittlement is the main reason for low-stress fracture initiation. Residual stresses enhance the brittleness in so far as they decrease the ductility by prestraining the tip of the notch.

PROPAGATION OF BRITTLE FRACTURE

The propagation of brittle fractures can easily be studied with such a device as proposed by Robertson. But propagation studies require a specimen of sufficient length. We use in Ghent a home-made 600-ton tensile machine, capable of testing 3-ft wide specimens (Figures 18, 19). It consists of three separate identical parts, each developing 200 tons so that it is possible to vary the external load along the expected fracture path (Figures 20, 21, 22, 23).

Propagation of a crack is possible in specimens free of residual stresses, if the following conditions are fulfilled: (1) the temperature of the specimen must be lower than the transition temperature, and (2) a minimum stress on the order of 11,000 to 17,000 psi. must exist across the specimen.

If a test plate is uniformly cooled below the transition temperature and a tensile stress is applied across the plate, then the brittle fracture initiated by impact at one end of the specimen will stop if the stresses are below this critical value.

We conducted some Robertson tests with varying tensile stresses across the width of the specimens. A specimen was stressed in tension to 22,000 psi. across its two outer sections, while no load was applied across the middle section. (Figure 23) The brittle crack stopped as soon as it entered the non-stressed middle section. A similar result was obtained if the two first sections were stressed and no load was applied on the remaining one-third (Figures 22, 24). This shows clearly that a minimum stress must be available in order to propagate a brittle failure.

Now the question arises whether or not residual stresses act as external stresses. To investigate this, it is necessary to initiate a crack in a specimen in which previously large residual stresses are induced.

Very important work in this direction was done here at the University of Illinois.[11] Tapered slots were welded perpendicular to the edges of the wide plates, creating residual tensile stresses at the edges and compressive stresses in the central section. The external load decreased the compressive stresses in this central section to zero, so that the existing stress field was quite comparable to the one we obtained in our specimen. But in our test the stress distribution was solely created by the external load. In the tests at the University of Illinois, although speed of fracture decreased, the fracture crossed almost completely the region with compressive residual stresses, contrary to our tests where the fracture stopped completely when entering the non-stressed section. This proves that there exists a fundamental difference between residual stresses and external stresses in the phenomenon of propagation of brittle fracture. This phenomenon can be explained by the fact that in the case of external loads, the load on the uncracked part of the specimen remains unchanged, while the distribution of the residual stresses change continuously during the propagation of the crack.

Brittle fractures are able to enter regions with compressive residual stresses and run quite a distance in to them, as was proven by tests on welded disks. These disks consisted of a 16-in.-diameter specimen with a 4 1/2-in. square opening in the center and welded into a 4-ft-diameter ring.[12,13] Due to the heavy constraint the inner disk underwent severe plastic deformation during welding. This resulted in high residual tensile stresses in the central part, balanced by compressive residual stresses in the outside of the disk. Local cooling of a small area at the corners of the square hole with liquid nitrogen led to initiation of spontaneous brittle fractures running radially toward the periphery in the zone of residual compression stresses. From these tests we can conclude that residual stresses may have an influence on the propagation of brittle fractures.

All these tests seem to indicate that little or no use of compressive residual stresses can be made in structures in order to avoid catastrophic failures. More possibilities seem to exist in the exploitation of the ductility of the metal by adequate heat treatment. Local heating with oxyacetylene torches gave satisfactory results in several tests. Running brittle cracks stopped in these spot-heated areas, although they are regions where high residual tensile stresses were set up. This underlines clearly the importance of the ductility of the metal versus the presence of residual stresses (Figure 25).

Figure 26 shows the result obtained in a cruciform test piece. It is known that in these specimens a brittle fracture propagates along the diagonal while a ductile fracture results in tearing off one arm of the test piece. Test-specimen B4 of Figure 26 was heated in its center by means of an oxyacetylene torch. The brittle fracture which was initiated in the angles of the cross stopped in the heated zone; this test convinces us that the residual stresses play only a secondary part in the phenomenon of brittle fracture propagation. As a matter of fact, by heating the center of the cruciform test piece residual tensile stresses have been set up in this zone and are compensated by compression stresses in the non-heated zone. The brittle fracture arrested in the zone of the residual tensile stresses which definitely proves that the deformation capacity of the steel is more important than the residual stresses.

CONCLUSIONS

I have tried to prove in this talk that the part played by residual stresses in the phenomenon of brittle fracture has been greatly exaggerated.

Concerning the initiation of brittle failures, we are convinced that the brittleness of the material at the tip of the notch is the predominant factor of the phenomenon. Such local embrittlement around notches, cracks, or discontinuities is caused by simultaneous straining and heating at temperatures between 150° to 350° C., and low average applied stresses may be enough to exhaust rapidly the remaining ductility and to initiate a catastrophic failure.

With regard to the propagation of brittle failures, we can say that even if tensile residual stresses facilitate the propagation, then we may not yet conclude that compressive residual stresses might form effective crack arrestors. I believe that local thermal treatments should prove to be much more efficient.

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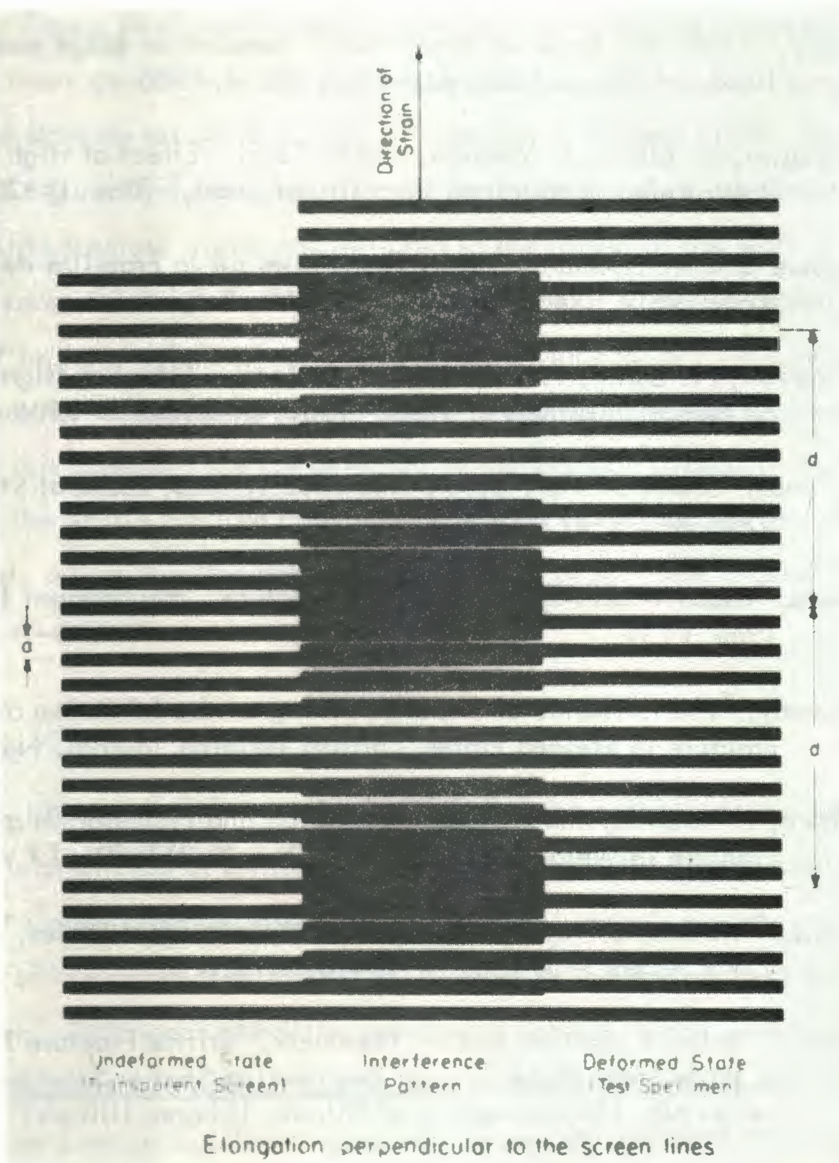

Figure 1 Principle of the Moiré technique (A. Vinckier)

Figure 2

Moiré pattern in a tensile specimen with a hole (A. Vinckier)

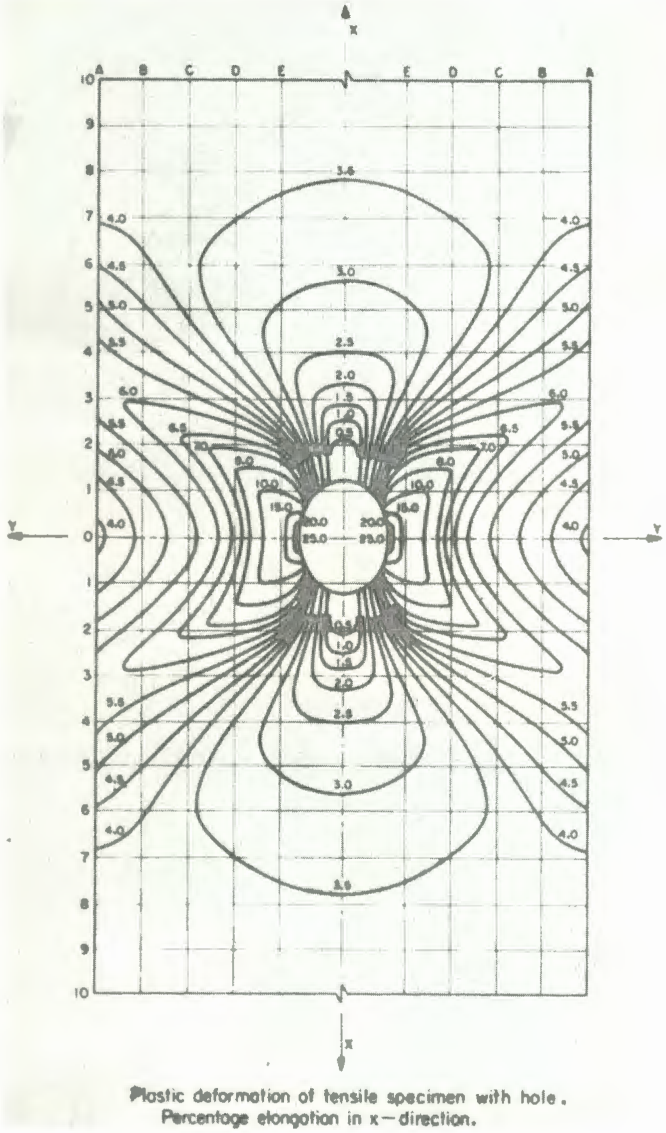

Figure 3

Iso-strain curves obtained with the Moiré pattern (A. Vinckier)

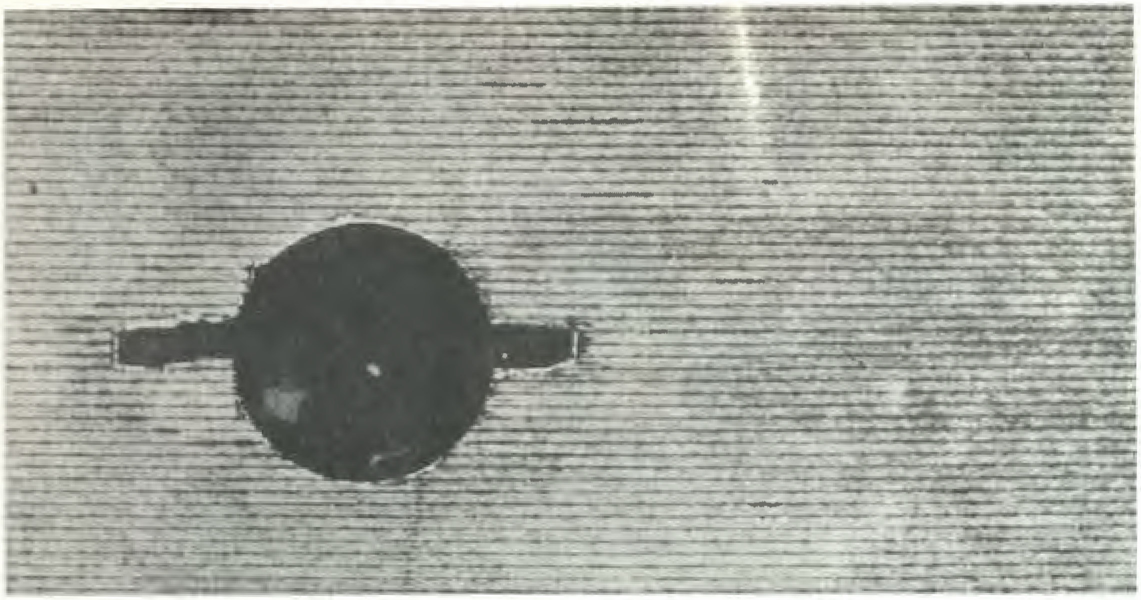

Figure 4 Detail of hole and saw-cut in a tensile specimen. The fatigue crack can not be seen in the unloaded specimen. Note the grid on the specimen. Distance of the grid lines 1/65 cm (R. Dechaene)

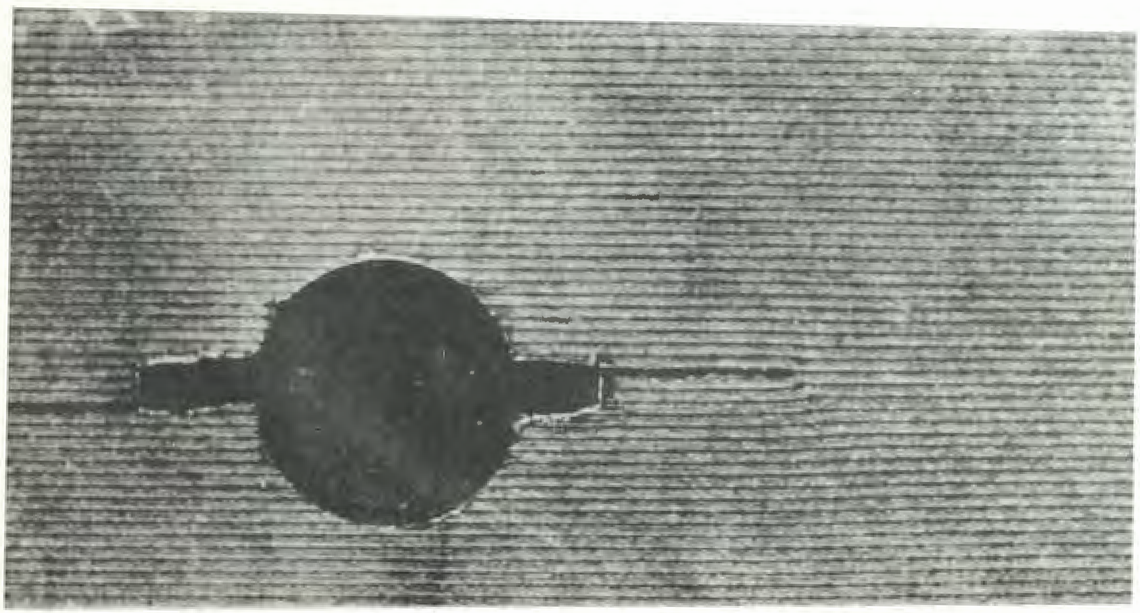

Figure 5 The same specimen slightly loaded in tension. Note the fatigue crack and the deformation at the bottom of this crack (R. Dechaene)

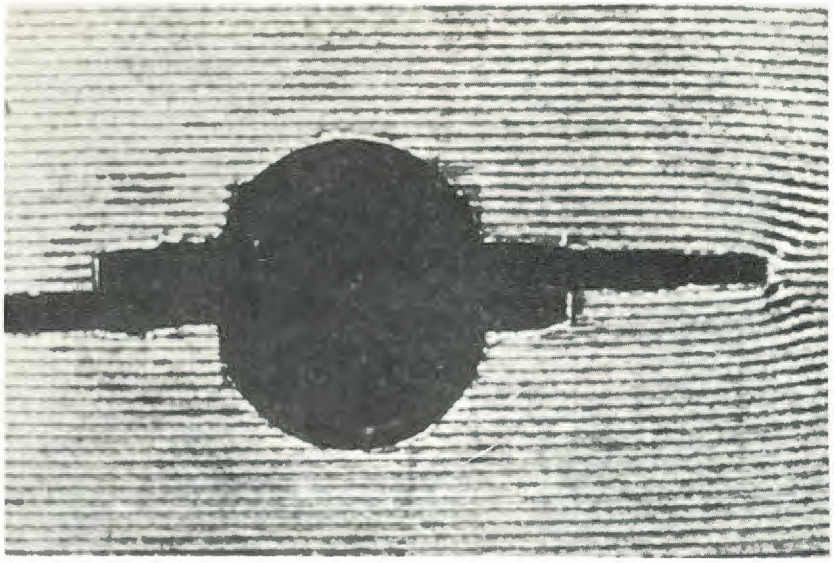

Figure 6 The same specimen at a higher tensile load. Note the severe plastic deformation at the bottom of the crack.

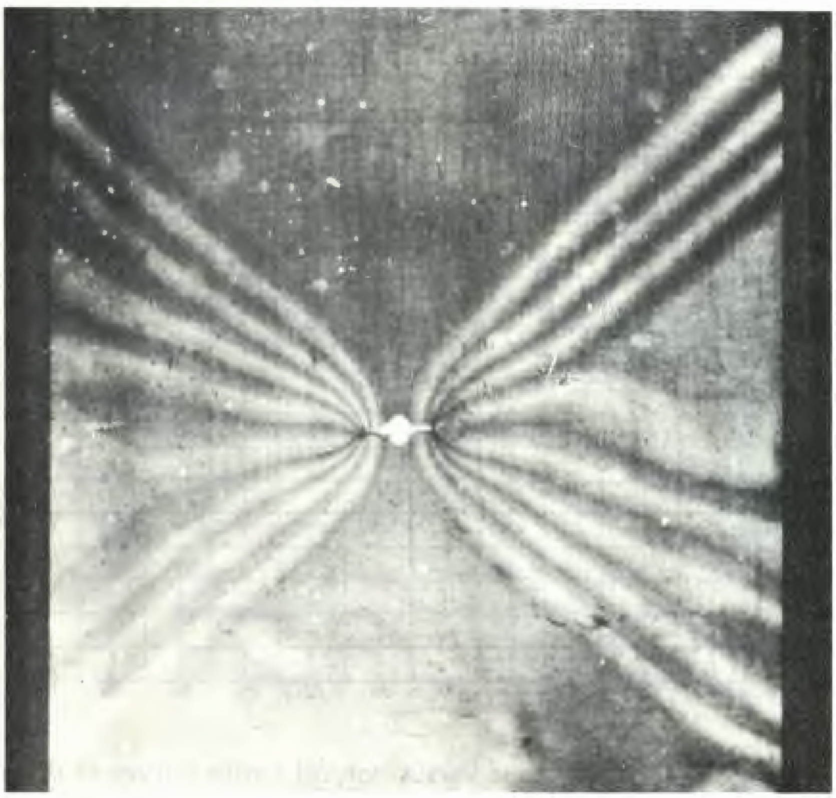

Figure 7 Moiré pattern of the same specimen (R. Dechaene)

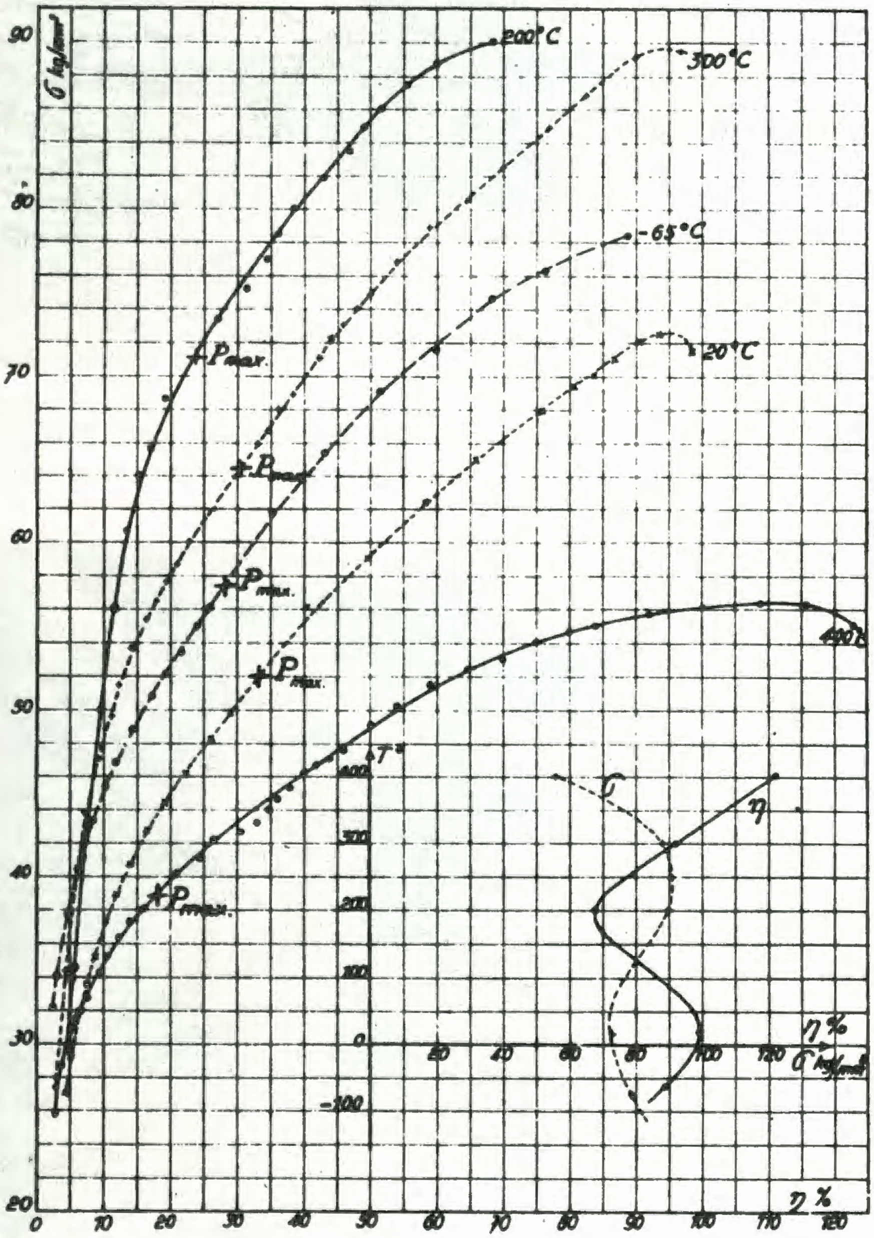

Figure 8 True stress versus natural strain curves at different temperatures of an unnotched specimen.

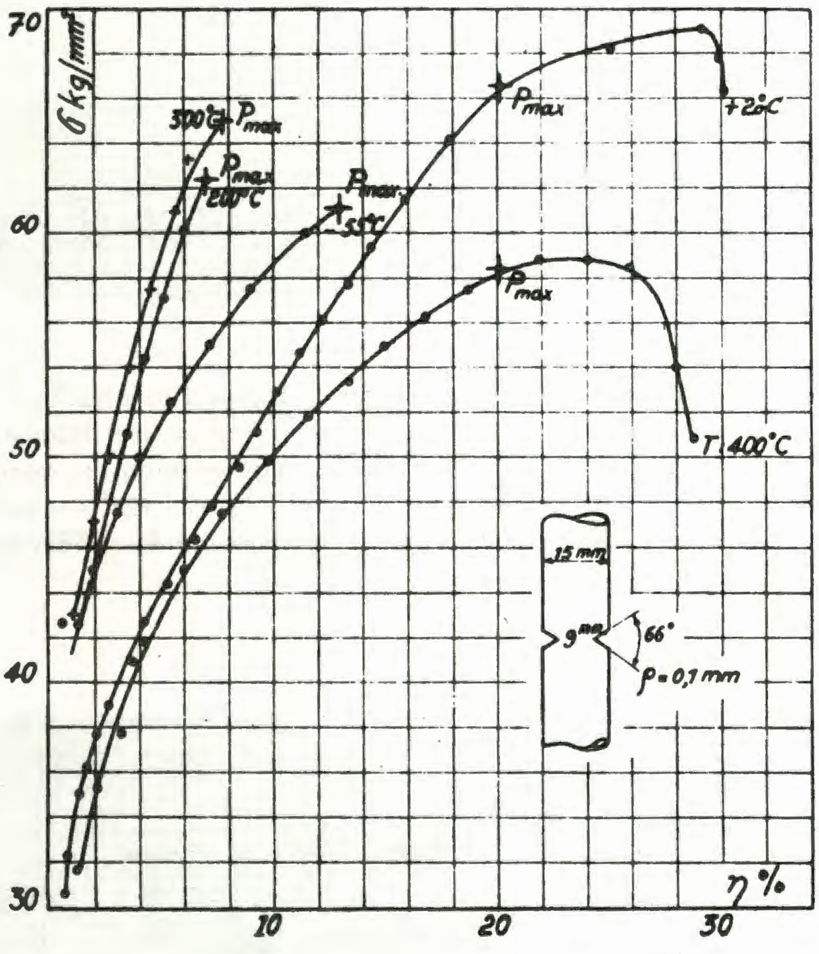

Figure 9 True stress versus natural strain curves at different temperatures of a notched specimen.

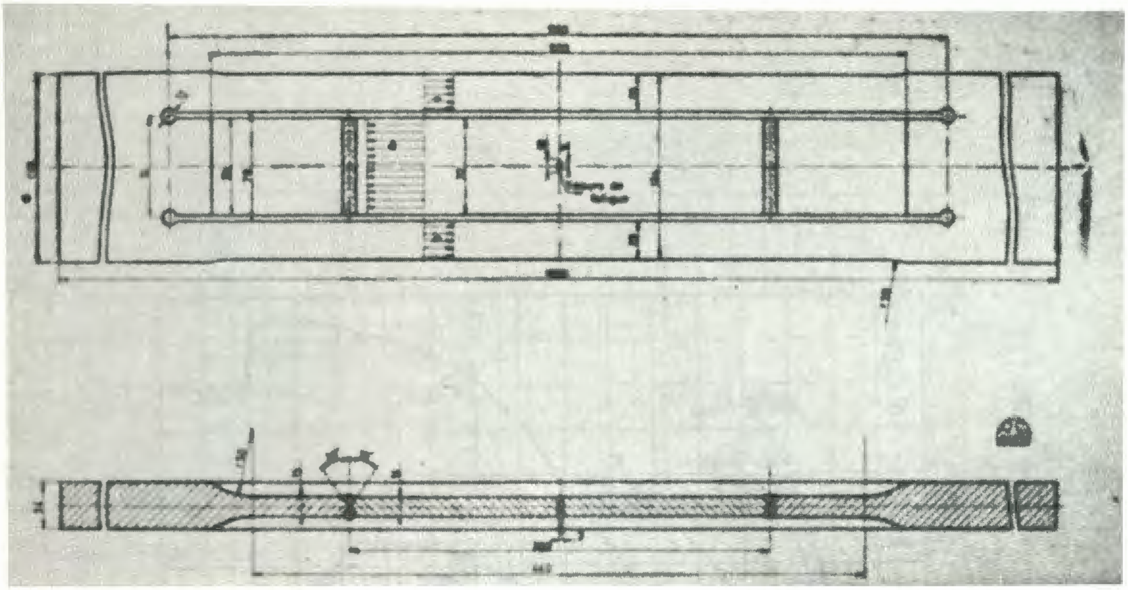

Figure 10 Specimen with symmetrically disposed longitudinal slits and with a fatigue crack in the central ligament. These specimens were obtained by welding the central ligament in the frame. By welding, residual stresses were set up in the central ligament, without heating the cracked zone.

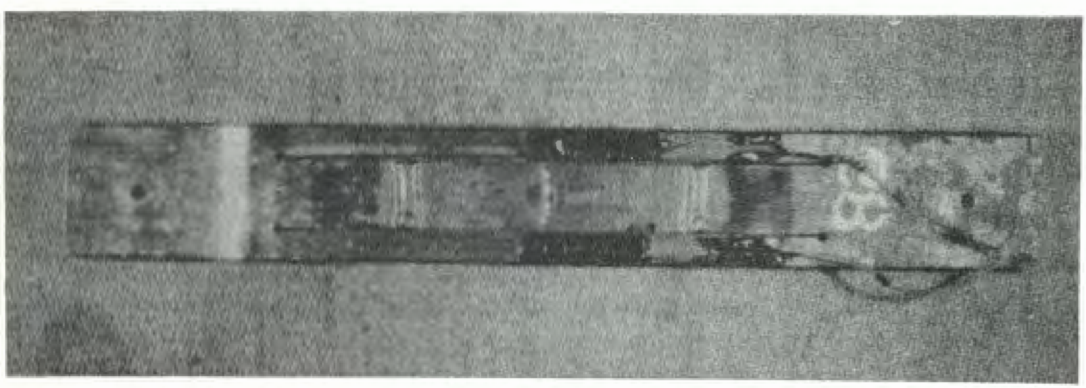

Figure 11 Photograph of a specimen as indicated on Figure 10.

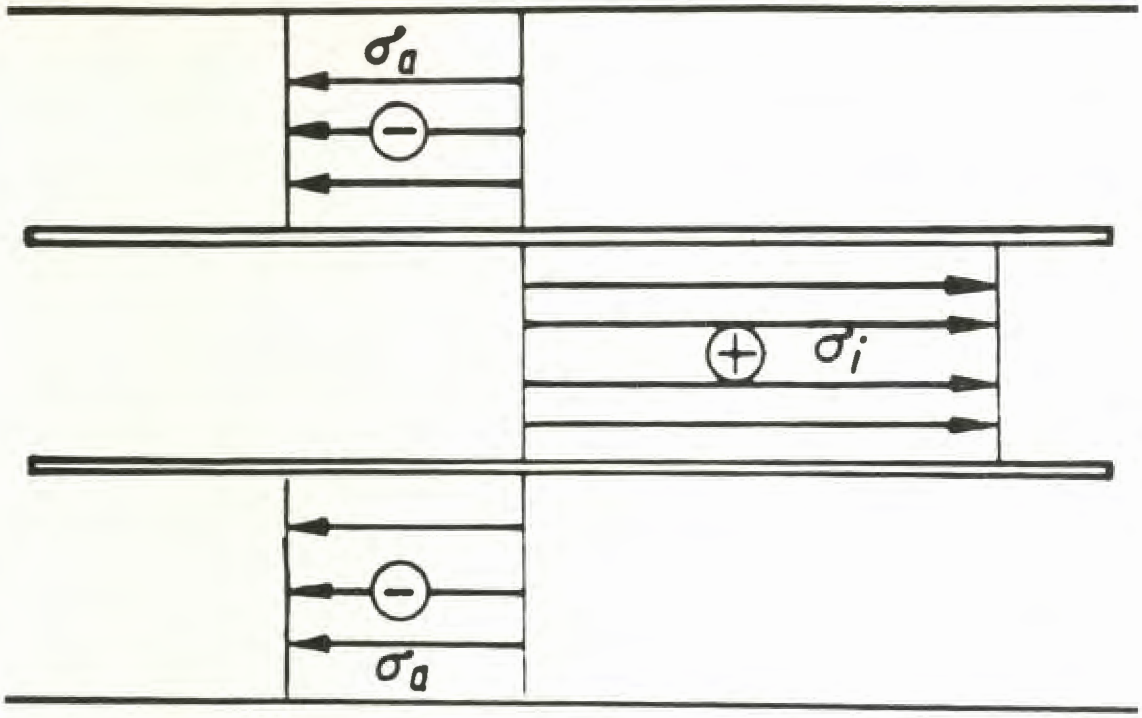

Figure 12 Schematic representation of residual stress distribution in the specimen with longitudinal slits, but without the fatigue crack in the central ligament.

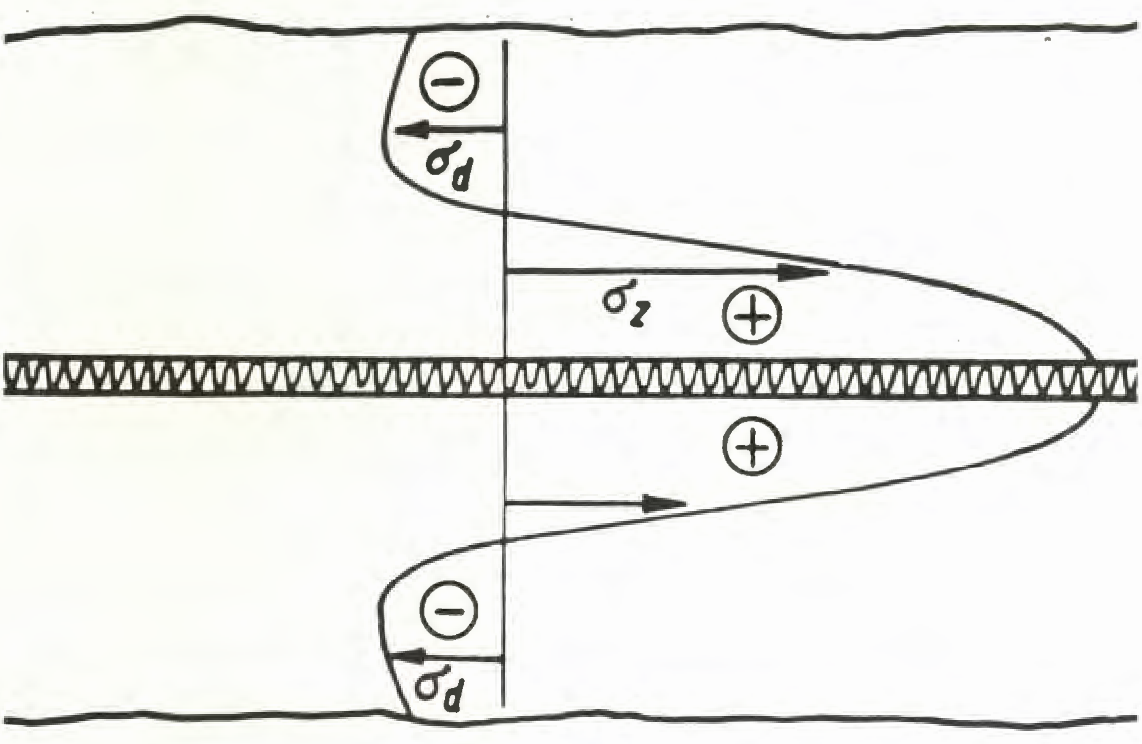

Figure 13 Schematic representation of longitudinal residual welding stresses in a butt weld.

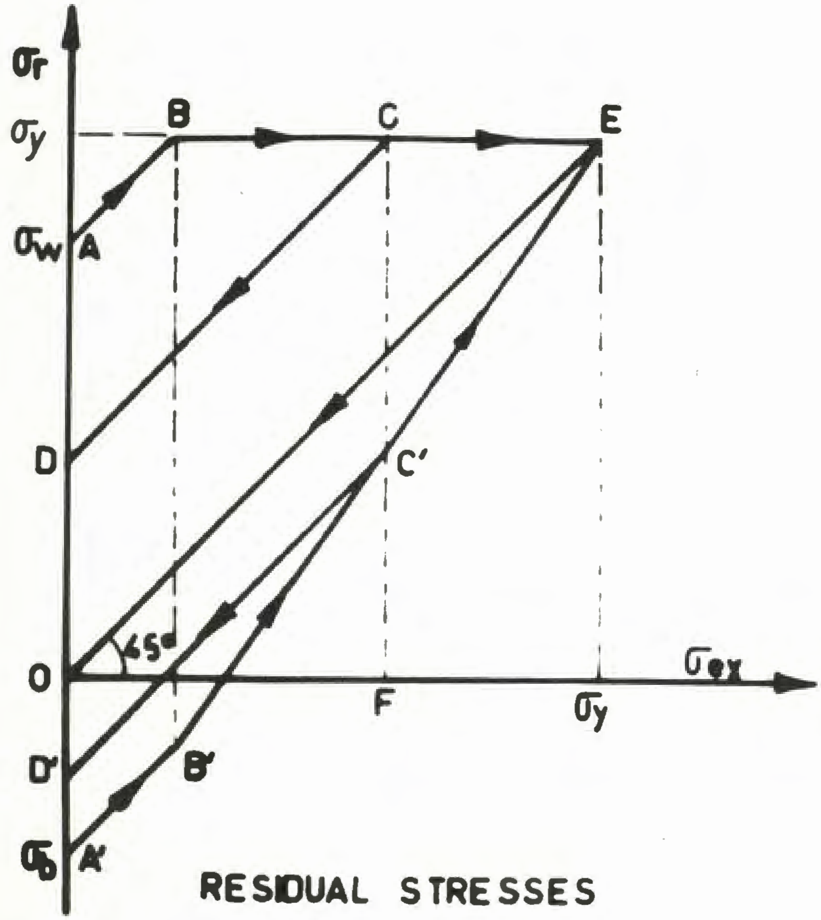

Figure 14 Theoretical diagram giving residual stresses (in web and columns) versus the external applied stress

$$\sigma_{ex} = \frac{P}{A}$$

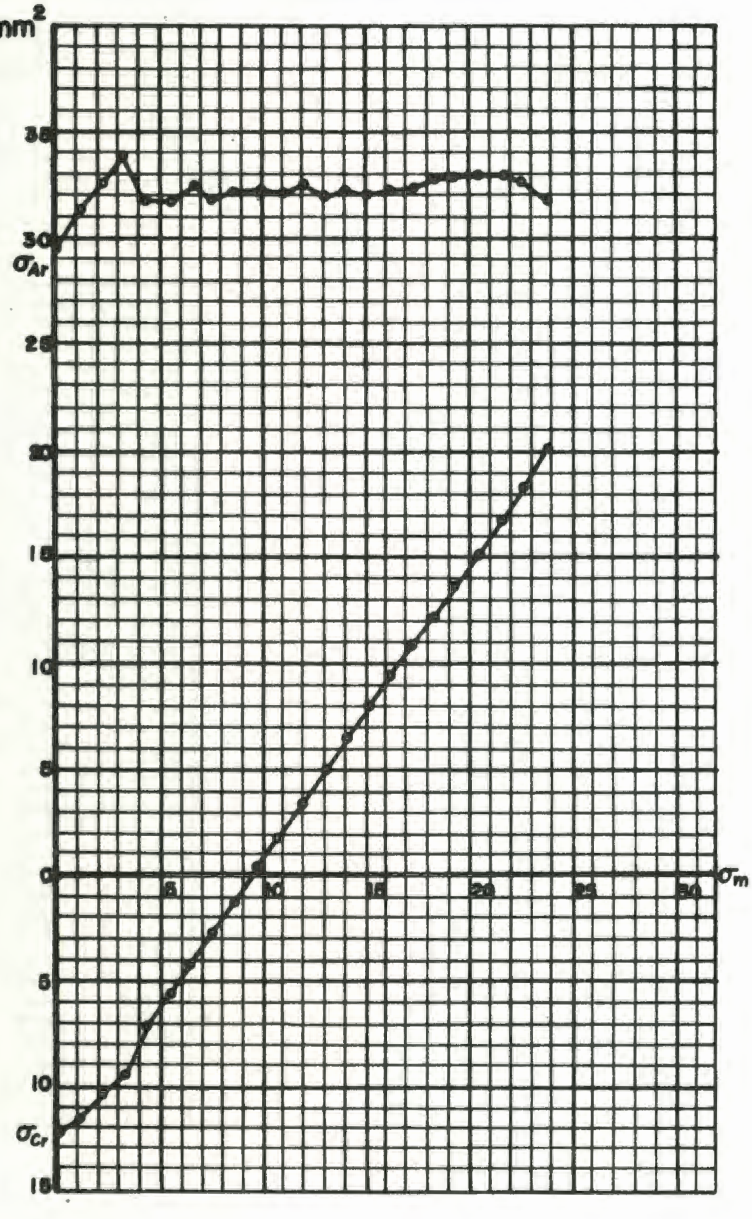

Figure 15 Experimental diagram of specimen 21 (Table I) The applied stresses reached 23.3 kg/mm^2, before brittle fracture of the web occured.

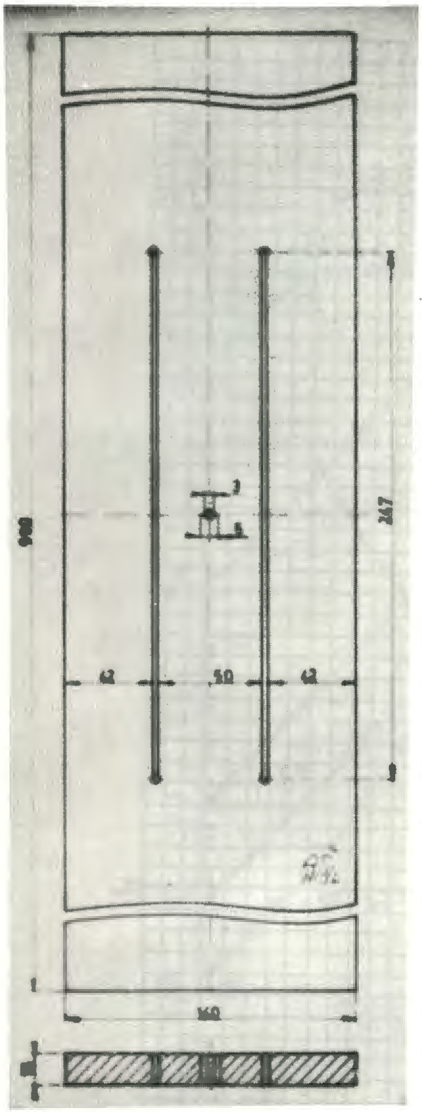

Figure 16

Specimen with symmetrically disposed longitudinal slits, and with a fatigue crack in the central ligament. These specimens had no welds; residual stresses were set up by heating of the central ligament.

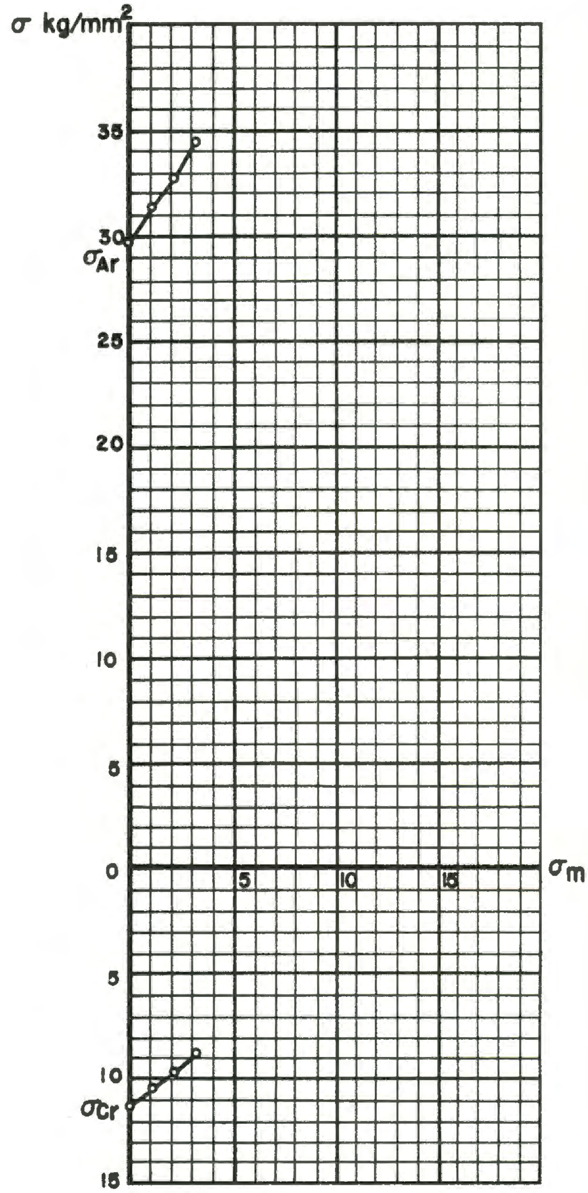

Figure 17

Experimental diagram of specimen 24 (Table III) The applied stress reached only 3.3 kg/mm^2 at the moment of brittle fracture of the web.

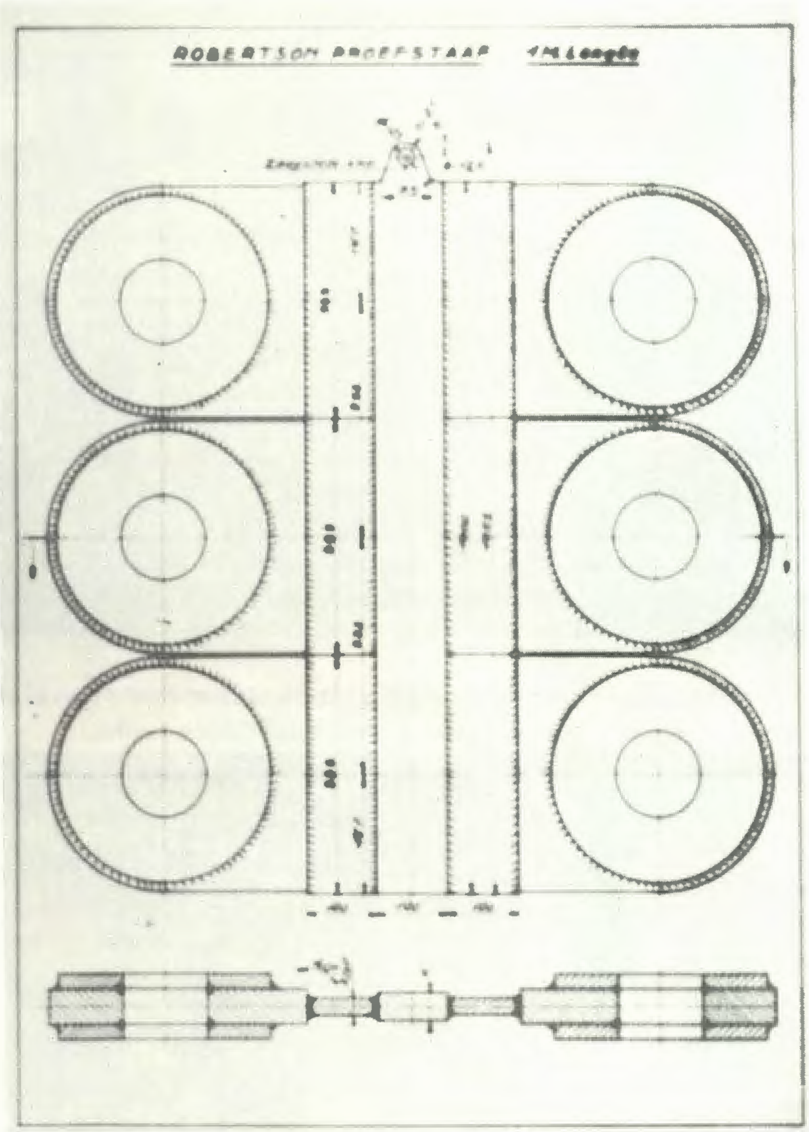

Figure 18 The 1-m wide plate specimen for the study of crack propagation according to Robertson's testing device.

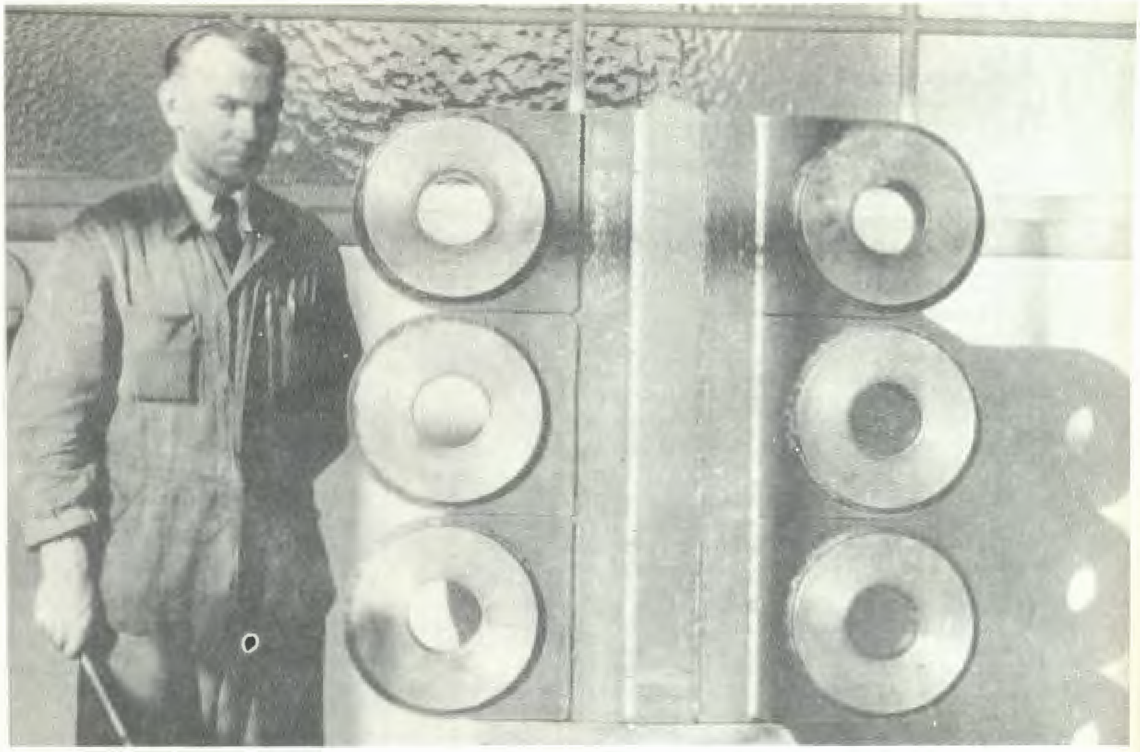

Figure 19   Photograph of specimen in Figure 18.

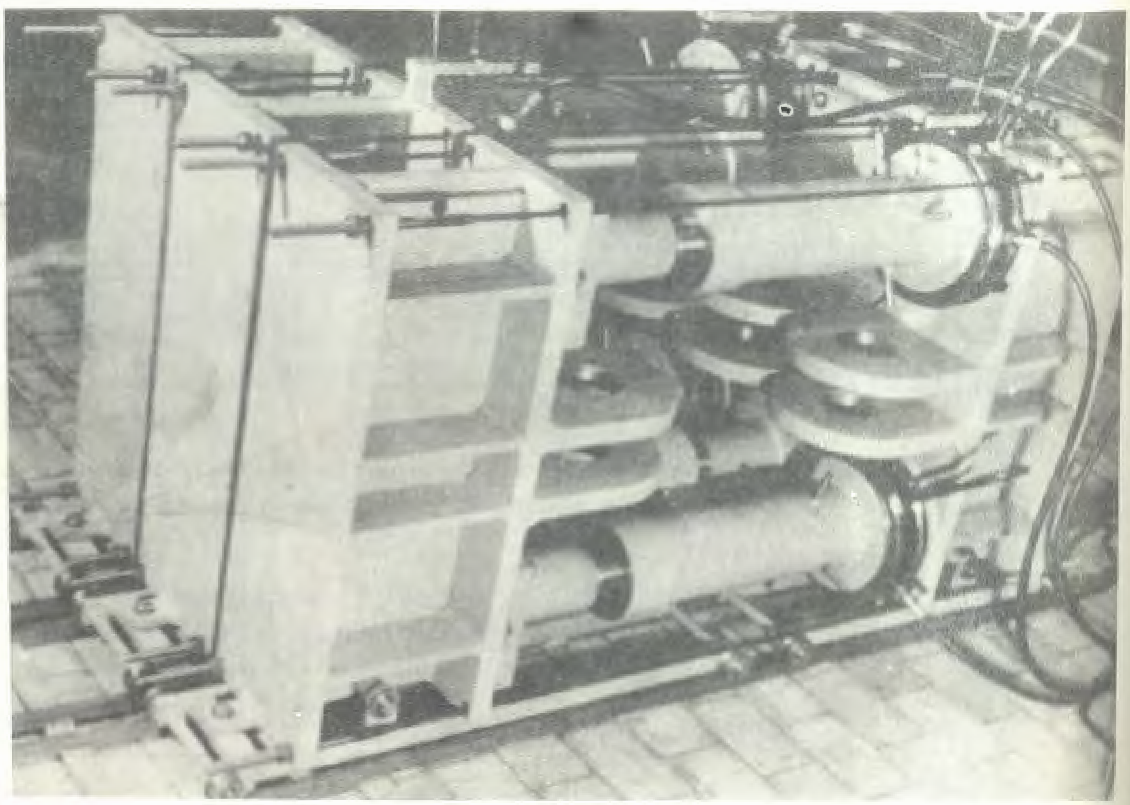

Figure 20   The 600-ton tensile machine, composed of three elements each capable of developing 200 tons.

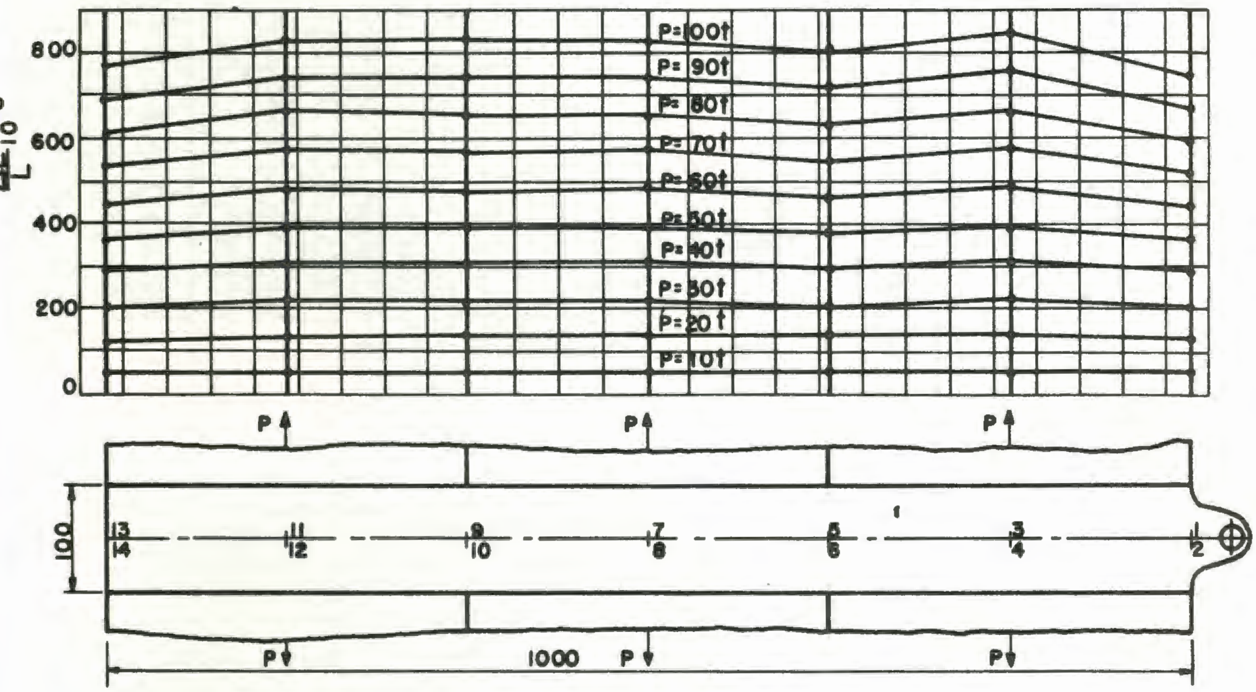

Figure 21 Measured strain distribution in a wide plate test when equal loads are applied to the three elements.

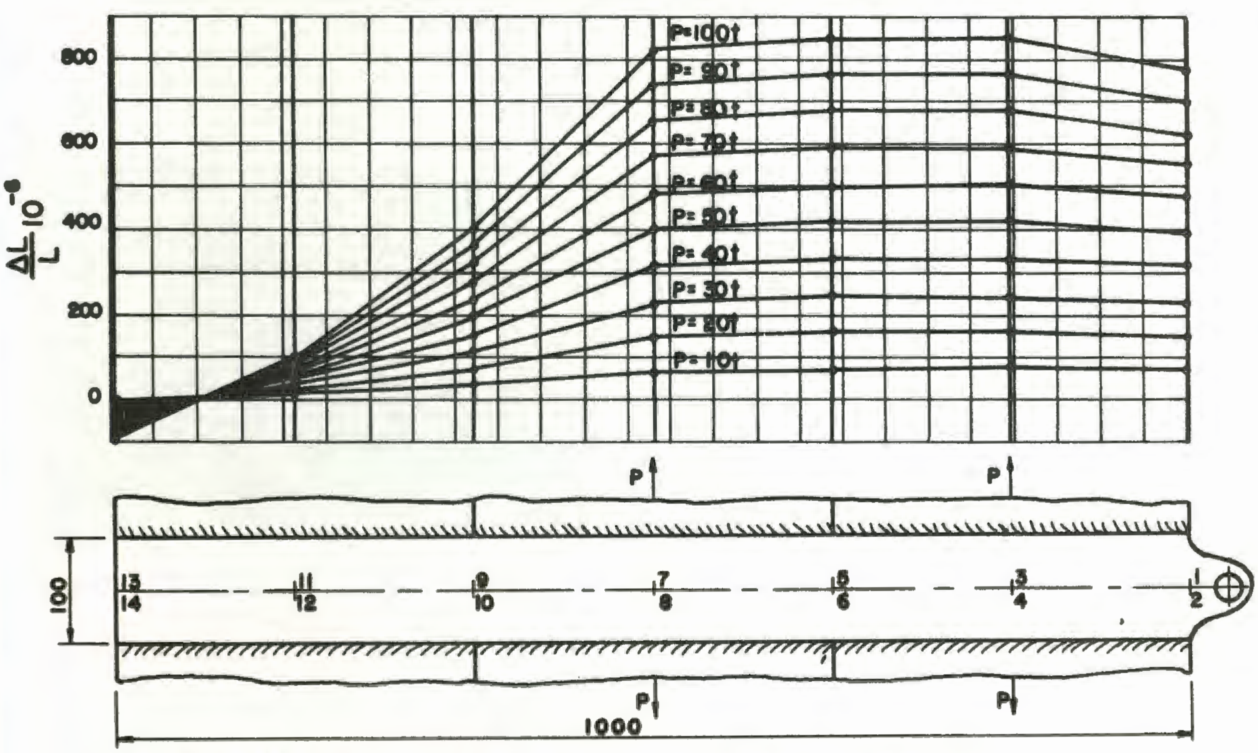

Figure 22 Measured strain distribution when loads are only applied to the central and one external element.

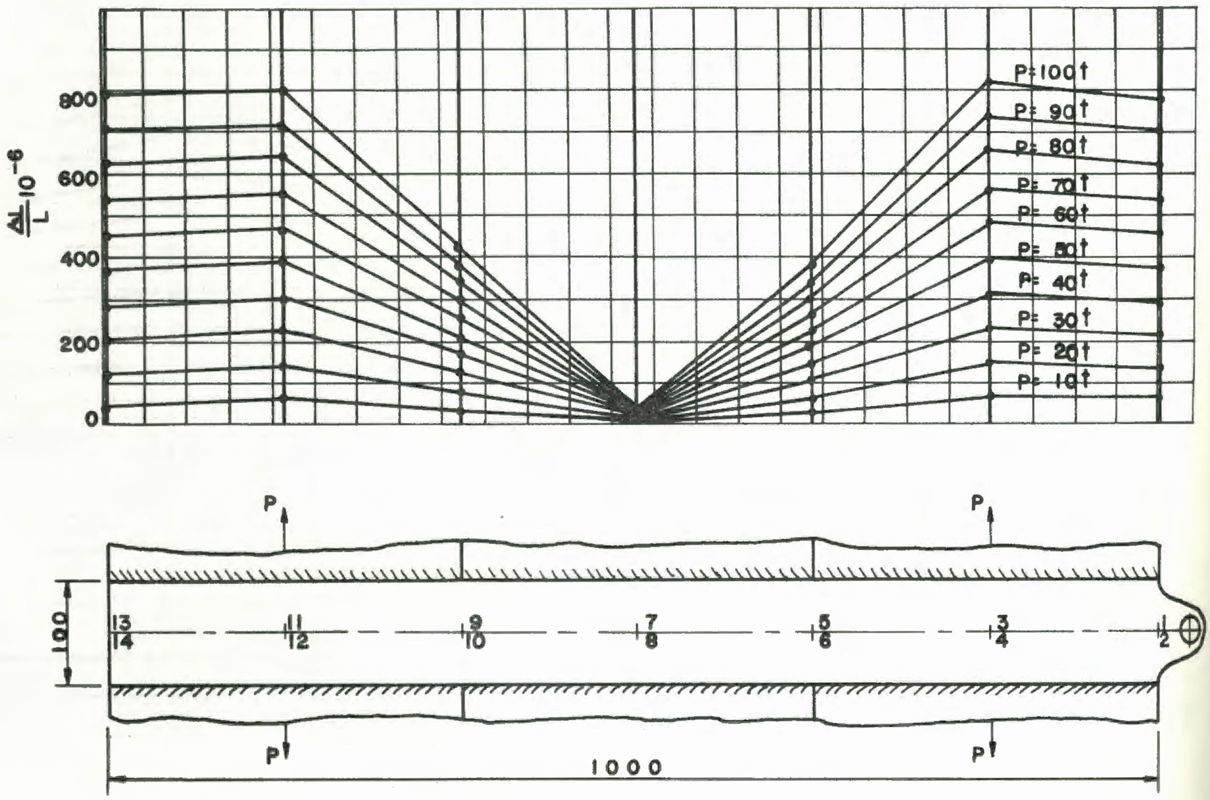

Figure 23 Measured strain distribution when loads are only applied to the two external elements.

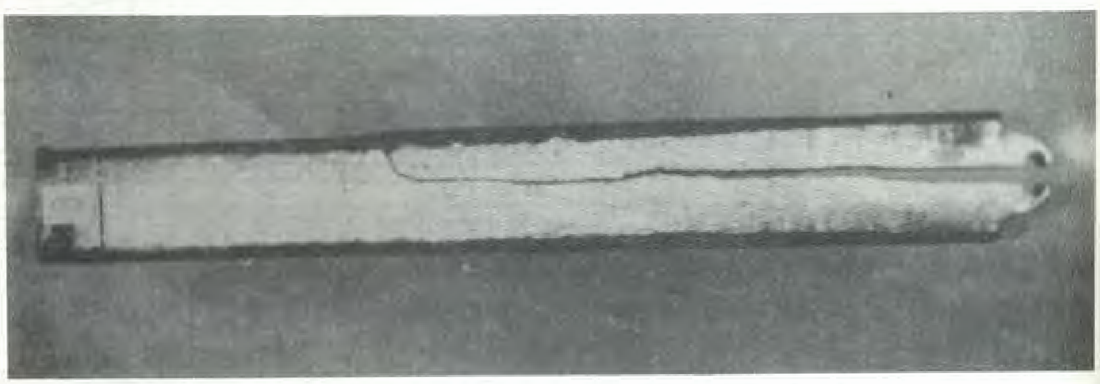

Figure 24 Robertson wide plate test. Load was applied to two first elements and no load was applied on the third element. Note the crack arrest as soon as the crack reaches the non-stressed section.

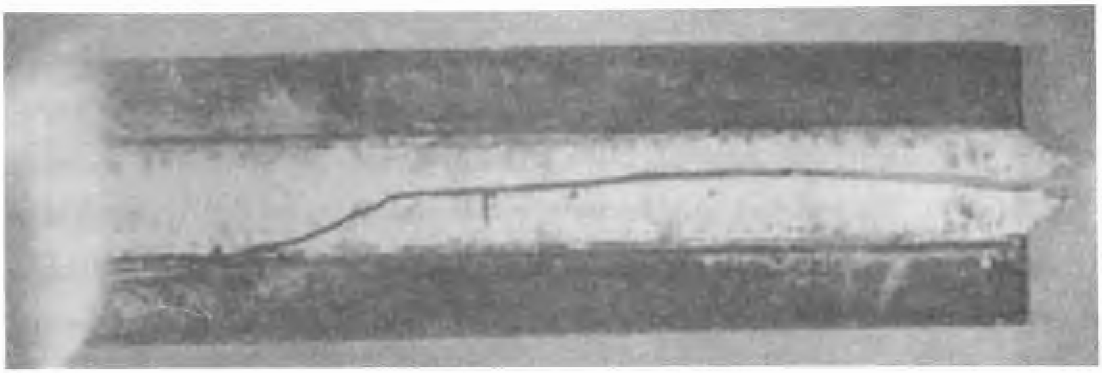

Figure 25 Robertson test on a wide plate. Note crack deviates in zone where local heating was applied.

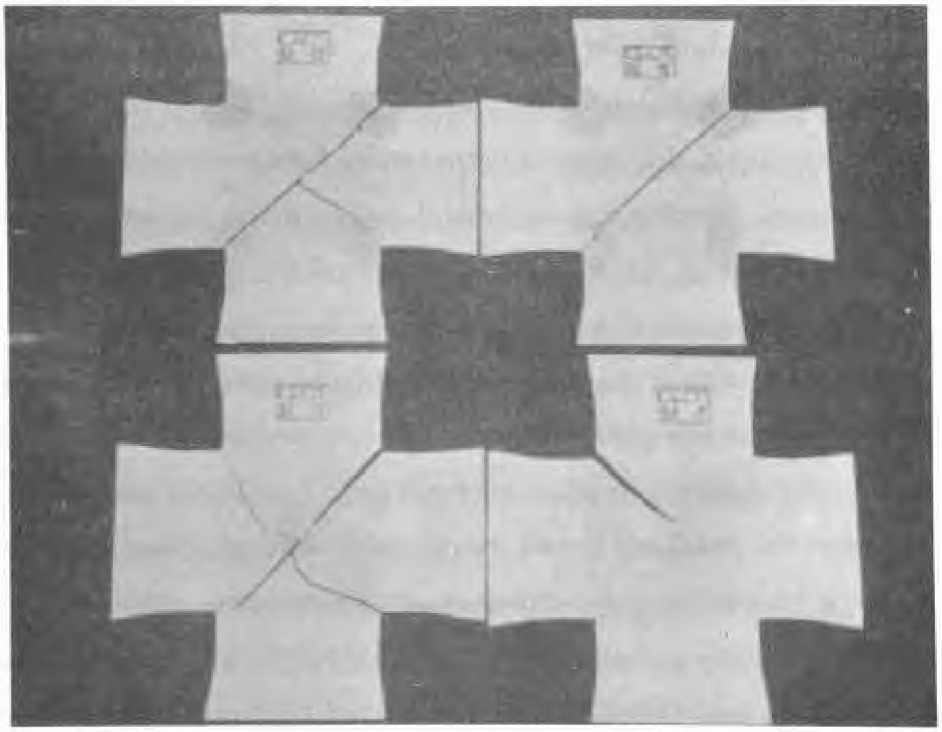

Figure 26 Brittle fractures in cruciform specimens. In the two upper specimens (No. B1 and B2) the corners, the initiation points of the specimens, were locally heated. In the two specimens at the bottom (B3 and B4) the central parts of the specimens were locally heated. Note the arrest in the heated zone (specimen B4) region of high residual tensile stresses. (The heated spots are slightly shadowed.)

RESIDUAL STRESSES

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PART I - MAGNITUDE AND EFFECTS OF RESIDUAL STRESSES

The welding point, moving along a plate to set down a weld bead, is heated to the melting temperature, while the surrounding material is heated to temperatures between that and room temperature. Those parts which have been heated to 300° C. or more will be plastically compressed due to the resistance of the surrounding cold material. When the hot metal cools, it has been so extensively compressed that it is too small and cannot shrink enough elastically, and therefore must shrink plastically. As a result stresses are imposed in the hot material equal to the yield point of the material. After complete cooling, we get tensile residual welding stresses of the yield point magnitude in the welded bead and in the near vicinity. In the parts further away from the weld the stresses are compressive stresses. Because the weld metal shrinks in the transverse as well as in the longitudinal direction, tensile residual stresses of the yield point magnitude are also set up in the transverse direction in and near the surface, if the parent metal is thick enough to withstand the stresses. If that is the case we obtain balancing residual compressive stresses in the material beneath the surface and weld bead.

If the parent metal is not thick enough or if, for other reasons, it cannot withstand the shrinking forces, we get quite different stress characteristics. In the following I am going to show some results which I have obtained when measuring residual welding stresses, and I will divide the different types of stresses into surface stresses, subsurface stresses, and triaxial stresses. I shall begin with surface stresses in pressure vessels.

SURFACE STRESSES

In the welding of pressure vessels there is sometimes a limit set with respect to the requirements for heat treatment after welding at a 3/4-in. wall thickness. Pipes and vessels of thinner wall thicknesses do not normally need stress annealing. The question arises whether this limit is correct. It is based on experience, and probably to no small extent on guesswork. In the regulations no account is taken of the size of the vessel or pipe, which in all probability influences the magnitude of the residual welding stresses. In the following I shall present information about the surface residual stresses in vessels of different diameters.

The residual welding stresses were measured in the longitudinal welds of vessels and, in some instances, in the transverse welds and also in the transverse welds between the casing and the end. The largest working stresses occur in the casing's longitudinal weld. Attention will first be devoted to the transverse stresses in this weld.

Figure 1 shows the magnitude of these transverse stresses in a casing 3600 mm in diameter and 28 mm thick plate. Measurements were taken both at the center of the weld and in the material beside the weld. As will be seen, the residual welding stresses in the weld metal are of the same order of magnitude as the filler metal's yield point and decrease continuously to zero at about 30 mm. The filler metal was a low hydrogen basic electrode with a yield point of about 43 kg/mm^2 and the parent metal was mild steel. The joint was an unsymmetrical X-joint. Figure 2 shows the longitudinal stresses at the same points as the transverse stresses shown in Figure 1.

The measurements shown in Figures 1 and 2 indicate that residual welding stresses of the magnitude of the yield point are present along and across the welds in these large pressure vessels. The question now arises whether this is also the case in smaller vessels which have approximately the same plate thickness. A diagram is reproduced in Figure 3 with the vessel's diameter plotted along the horizontal axis and the residual welding stresses in the longitudinal weld's transverse direction along the vertical axis. The plate thickness varied between 27 and 32 mm. As may be seen, at large diameters the stresses approach the material's yield point asymptotically with increasing diameters of the vessels.

In small containers negative stresses are obtained instead; i.e., compressive stresses which appear to approach the diagram's vertical axis asymptotically.

In Figure 4 all measured transverse stresses in the longitudinal weld of casings with different plate thicknesses are set off, whereby the plate thicknesses are denoted by different symbols. Apart from a number of extreme results which may be due to various factors such as different cooling conditions following the change of electrodes, etc., or to some small differences in the method of welding, or to actual measuring faults, the measuring results obtained agree fairly satisfactorily with the plotted curve, which is the same as in Figure 3.

The investigation points to the fact that the residual welding stresses in the transverse direction of the pressure vessel's longitudinal weld are independent of the plate thickness, but increase continuously with the vessel's diameter from high compressive stresses to tensile stresses of yield-point magnitude.

The measurements thus indicate that no threshold value exists at a 3/4-in. plate thickness.

We have seen that the longitudinal residual welding stresses are tensile stresses of yield-point magnitude, but we have also seen that the residual welding stresses in the direction transverse to the weld can be anything between the yield point in tension or in compression, depending on the diameter of the vessel.

The residual stresses in welded ship hulls are also of very great practical interest. In order to investigate those stresses, measurements were undertaken before launching on the welded hull of an 18,000-ton tanker, M/T Markland, which was lying on the ways in the shipyard. The measurements were made in places situated partly in the main deck and partly in the bottom.

In Figure 5 the directions and magnitudes of the residual welding stresses are indicated. The minus signs refer to compressive stresses; all other stresses are tensile stresses. The upper part of the figure refers to the main deck and the lower diagram to the bottom, the latter as seen from above. In both cases starboard is to the left and port to the right in the drawing. The longitudinal axis of the ship is marked out with a dot-dash line. The dashed lines indicate multi-run welds and the dotted line represents a weld made with the submerged arc welding method. The crosses indicate the directions of the principal stresses and the numerals give the stresses in kilos per square mm.

As seen in the figure, the stresses in the plates proper were rather small. This is due to the fact that the plates were stress annealed before welding. In all welds high stresses were measured, especially along axis of the welds. Generally, these are tensile stresses. The surprisingly high value of 52 kilos per mm^2 was found in a multi-run weld.

The investigation reveals that remarkably high stresses are present only in the welds and their vicinity.

Since the time of the measurements in Figure 5, the ship Markland has been sailing in freight traffic between the Persian Gulf and England. In January, 1958, the ship returned to the dockyard for repairs. The residual stresses were again measured, both when the ship was lying in the water and after it had been placed in a dry dock.

The measurements of the stresses were carried out at locations which lay close to the locations used in 1952. In Figure 6 a view is shown, on the left, of the ship as seen from above with the positions of the measurement locations marked by points. On both occasions in 1958 the measurements were made only in the main deck.

The results of the measurements may be seen on the right-hand side of the figure. Here, three magnifications of the measuring region in the ship's deck are shown The main stresses obtained are indicated by crosses and numerals. The numerals indicate the stresses in kilos per mm^2, the plus signs show tensile stresses, and the minus signs show compressive stresses. The top illustration reproduces the results obtained in 1952, while the two lower illustrations relate to measurements made during 1958. The welds are indicated by fishbone patterns.

From a study of the figure it will be seen that after welding, the ship had residual stresses in the welds of a magnitude equal to the weld metal's yield point. This was the case both with respect to the longitudinal stresses and the transverse stresses. After 6 years of service the residual stresses in the longitudinal direction of the welds had been reduced to approximately zero, whereas the transverse stresses had changed over to compressive stresses having a magnitude of 10 to 15 kilos per mm^2. Relatively high compressive stresses have been set up in the plates outside the welds. The loading conditions were apparently only slightly different when the ship was in the water and after the ship had been placed in dry dock.

SUBSURFACE STRESSES

The measurements just discussed were undertaken solely for the purpose of determining the residual stresses at the surface of the welds and deck plates. The residual stresses below this surface, the subsurface stresses, are probably influenced by the working stresses also. In order to investigate this matter, measurememts were carried out in the spring of 1958 of the residual stresses under the surface of the plates and welds in two ships, namely the M/T Kronoholm which was under construction and an older ship, the M/T Rudolf Andersson, which had been in service for five years. The measurements in these ships were made approximately in the same positions as the measurements noted earlier on the M/T Markland. Thus, in this instance it was not the same ship that was subjected to stress measurements, but two separate ships; nevertheless the fact remains that the results obtained fit in so satisfactorily with those obtained from the M/T Markland that it may be stated with a reasonable degree of certainty that the residual stress conditions are identical on all ships of this type.

The results of the measurements obtained on the M/T Markland, the M/T Kronoholm and the M/T Rudolf Andersson have been combined in diagrammatic form in Figure 7. There are three diagrams on the left and three on the right. Those on the left relate to stresses in the longitudinal direction of the welds and those on the right in the transverse direction. Points indicate stresses in a new ship and crosses the stresses in an older ship. The four upper diagrams indicate the stresses at two points which are located symmetrically in relation to the center line of the ship and occur in welds across the ship. The two lower diagrams indicate the stresses at a point in a weld which runs along the ship. The results are fairly well distributed around the plotted curves and lines, although incorrect measurements have apparently been made at some levels of thickness.

From the lines and curves in Figure 7 it may be seen that in a new ship the longitudinal stresses decrease linearly towards the interior of the weld metal. This is also the case initially with the transverse stresses which reach a minimum, however, at half the plate thickness; zero or small compressive stresses then exist at a greater depth in accordance with the dotted parts of the curves. Measurements at a depth greater than about 20 mm could not be carried out with satisfactory accuracy.

In an older ship the measurements exhibit an entirely different state of stress. From zero or compressive stresses at the surface, both the longitudinal stresses and the transverse stresses rise very rapidly in a layer which is only a few millimeters in thickness. After reaching a maximum the stresses decrease, and the curves for the multi-pass welds run parallel to some extent with the curves for a new ship. The stresses are greater, however, in an old ship after a maximum has been reached than in a new one. The longitudinal stresses decrease more rapidly than shown by the initial linear lines.

The longitudinal stresses in submerged arc welds (Figure 8 upper part) exhibit the same character as the multi-pass welds, but the transverse stresses are very low at the surface in a new ship and rise under service conditions to a high maximum at half the plate thickness where the stresses in the multi-pass welds are at a minimum. Owing to the structure of the submerged arc weld it is more difficult to obtain reliable measured values of residual stress in it than in the multi-pass welds.

The residual stresses are predominantly positive (tensile) under the surface of the material in a longitudinal and a transverse direction both in an old and new ship. On account of the establishment of equilibrium the surrounding parts of the material in the immediate vicinity are subjected to negative (compressive) stresses.

Attention may be drawn to the fact that the effective reduction in stress at the top of the welds produced under service conditions has taken place in a relatively thin layer where a condition of compressive stresses has been set up. This is apparently something that has occured in the whole surface of the deck plates, judging by the results obtained at points located in the deck plates themselves far beyond the welds. The bottom of Figure 8 shows the results of measurements made at such points in an old ship. As may be seen, a very thin surface layer exists here also, in which the stresses have become compressive stresses. According to the results the stresses in the surface layer of the deck plates themselves were practically zero in a new ship.

Apparently a general condition of residual stresses has been superimposed on the original condition of stress at the surface of the deck plates, while at the same time the tensile stresses have increased in the parts below the surface owing to the establishment of equilibrium. It is therefore the surface layer which is the primary cause of this superposition of the residual stresses. A difference of opinion may exist concerning the reason for this behavior in the surface parts of the plates. It has been explained by the statement that when the ship sails in tropical waters the deck becomes strongly heated by the rays of the sun. Because of the heating it is necessary to cool down the deck plates from time to time by flushing them with water. This results in a rapid shrinkage in the actual surface layer which is cooled before the underlying parts of the material; and, therefore, with the shrinkage of the latter, it must be stressed beyond the material's yield point. Consequently, after the equalization of the temperature, the compressive stresses as well as the equilibrium will require balancing tensile stresses in the interior parts of the material.

We have seen that in a weld made in several runs we get tensile stresses in the surfaces but compression stresses between the two surfaces. That case is confirmed by Figure 9, which shows the stresses in a weld made in many runs between two plates. The welding was performed in fully unstrained condition. Figure 10 shows the stresses in an analogous weld, but the weld is here made in only one run with the submerged-arc method. The high tensile stresses beneath the surfaces are different than in Figure 9. The reason for the different stress conditions in these two welds is the difference in manner of cooling.

TRIAXIAL STRESSES

Tensile residual stresses are found at certain points inside a material. Inside a weld the longitudinal, transverse, and perpendicular stresses can be of the same magnitude. Figure 11 in all three directions shows an example of that kind. Two plates have been welded together in fully free conditions, without restraint. The thickness of the plate was 25 mm.

It will be seen from the diagram that the longitudinal and transverse stresses in and close to the top and root of the weld are tensile stresses. Inside the weld there is a region somewhat below the center under compressive stresses in all three directions. This point serves as a moment point for the shrinkage stresses in the upper part of the weld, when the latter has been completed and cooled down. At the same time tensile stresses are produced at the root in the transverse direction. The longitudinal stresses are shown to the left, the perpendicular stresses to the right, and the transverse stresses in the middle.

Another example is shown in Figure 12. The method of welding was the same as in the previous case but the plate was thicker, namely 38 mm. The longitudinal stresses are shown at the upper part of the figure, the transverse stresses in the middle part, and at the bottom the measured perpendicular stresses are plotted. We see a distribution of the residual stresses similar to that in the thinner plate.

It has been found in practice that a risk of crack formation exists in the direction of the bisectors when welding is undertaken in such a way that two welds intersect each other, if we weld the intersecting weld last (Figure 13 upper left), but not if the intersecting weld is done before the passing weld is complete (Figure 13 upper right). The case is similar when joining two beams (Figure 13 lower part).

Measuring the stresses in the case shown in the upper left of Figure 13 was undertaken in points shown in Figure 14, namely in points on lines perpendicular to the bisectors of the angles. In Figure 15 the stresses are plotted on zero, 5-, 10- and 15-mm depths from the top surface. The stresses are denoted σ_a for the longitudinal stresses, σ_b for the transverse stresses, and σ_p for the perpendicular stresses. No very high residual stresses exist. A typical example of the stress distribution at a point on the bisector is given in the diagram in Figure 16, where the depth from the top surface of the plate is set off on the horizontal axis and the stresses σ_a, σ_b, and σ_p on the vertical axis.

Many plates were welded and tested in the way shown in the upper left of Figure 13, and among them there was a weld that showed a quite different stress distribution. We see the stresses in Figure 17. Here a clear risk of fracture is present. In the middle part of the plate thickness the stresses are nearly equal in all three directions and we know from the von Mises hypothesis that in such a case the material is very brittle. The stress distribution very clearly indicates that a crack can be initiated in the direction of the bisector, which confirms the practical experience.

THE EFFECTS OF THE RESIDUAL STRESSES

Static Strength. We may begin by examining whether the longitudinal residual stresses in and close to a weld reduce the static strength in the region subjected to stress. It may be assumed that the region in question is replaced by a framework as shown in Figure 18, in which the central bar corresponds to the weld metal, and the two other bars represent the parent metal. The ends are assumed to be entirely rigid so that on the application of a load, deformation will only occur in the three bars. The sectional area of bar no. 1 is denoted by A_1 and that of each bar no. 2 by $A_2/2$.

On tightening the nut a tensile force $+P$ is set up in bar no. 1. The framework will then have a residual stress which in bar no. 1 will be of the intensity

$$\sigma_1' = \frac{P}{A_1}$$

and in the bars no. 2 $$\sigma_2' = -\frac{P}{A_2}$$

If an external tensile force Q is applied to the framework, it will produce the tensile stress

$$\sigma_1'' = \sigma_2'' = \frac{Q}{A_1 + A_2}$$

The total stress in the bars will then be

$$\sigma_1 = \sigma_1' + \sigma_1'' = \frac{P}{A_1} + \frac{Q}{A_1 + A_2} \qquad \sigma_2 = \sigma_2' + \sigma_2'' = -\frac{P}{A_2} + \frac{Q}{A_1 + A_2}$$

If, as is the case in a weld

$$\sigma_1' = \sigma_s \quad (\sigma_s \text{ is the yield point})$$

Then

$$\sigma_1 = \sigma_1' = \frac{P}{A_1} = \sigma_s \qquad \sigma_2 = \sigma_2' + \sigma_2'' = -\frac{P}{A_2} + \frac{Q}{A_2}$$

Let Q_{max} denote the load at which the framework cannot withstand further loading but begins to yield in the bars no. 2, that is to say,

$$\sigma_2 = \sigma_s = -\frac{P}{A_2} + \frac{Q_{max}}{A_2} \qquad \text{Thus } Q_{max} = \sigma_s (A_1 + A_2)$$

A framework without residual stresses (P = O) can withstand the load:

$$Q_{P=0} = \sigma_s (A_1 + A_2)$$

Thus, the framework will withstand the same load both with and without residual stresses.

A very important circumstance, however, is that there are no stress raisers or notches, because in that case the central bar cannot yield without the stress in it increasing until fracture occurs. However, if there are slag inclusions, cooling cracks, or something like that in a weld the residual welding stresses may be dangerous, especially at low temperatures.

In order to obtain practical confirmation of the foregoing theoretical considerations, the following experiments were made.

Two plates of mild steel with the dimensions 280 x 250 x 12 mm were welded together with a 60° V to form a single plate. On the completion of welding, the weld and surrounding material were cut out by flame cutting so that a bar 50-mm wide with a weld along the center was obtained. The bar was cooled with water during the cutting process. It was machined on both sides and given a reduced section as shown in Figure 19. The bar was placed in a tensile testing machine and its strain read off with the help of a sensitive tensometer. A bar of the same shape taken from the parent material, but without a weld, was subjected to a similar tensile test and the result of the measurements on the two bars are reproduced in Figure 19 in the form of a complete stress-strain diagram and details of the same within the "Hooke" region. A curve is also plotted here for the weld metal in the form of an all-weld metal test specimen. The curves are indicated "Grundmaterial" for the parent metal, "Svetsskarv" for the specimen of weld and parent metal and "Helsvetsprovstav" for all-weld metal test specimen. "Förlängning" means strain and "Svets" means weld.

As may be seen, the welded bar does not follow Hooke's law. Yielding already occurs in the bar under the smallest load as indicated by the above theory. The yielding then only takes place in the weld metal, whereas the surrounding region which is under compressive stresses is relieved of the stress. If, after a given load has been applied, this load is removed, the bar will not return to its original length but will follow the dotted line and remains elongated after the complete removal of the load.

In the right-hand diagram it is seen that the final elongation for the welded bar lies between that of the stress-relieved weld metal and the parent metal.

It may be concluded from the diagram that although the residual welding stresses along the weld are at the intensity of the weld metal's yield point, the capacity for strain is not appreciably reduced. Nor have the yield point and the tensile strength of the weld been reduced, the latter being capable of withstanding as great an external load as the stress-relieved parent metal. It will be noted, however, that on the slightest tensile stress being set up in the direction of the latter, yielding and a permanent change in length will occur.

Impact Strength. We have now seen that if the weld is free from stress raisers like slag inclusions, notches, etc., the residual welding stresses are of no importance when the load is static at room temperature. We will now investigate what the effect will be if the load is an impact and the temperature is low. We will go back to the framework that we used in connection with static strength. In Figure 20 we see two yokes which are connected by two bars and between these is located a third bar which can be tensioned by means of a nut with a force P. The two yokes can be drawn apart by an external force until the central bar breaks at an external force Q_B, the bar's tensile strength. The break occurs after the bars have been elongated by the amount e_B.

The diagram in Figure 20 (middle left) shows the relation between the load and the elongation by means of the curve in broken lines. This curve may be replaced with satisfactory approximation by a straight line in accordance with the figure. According to the latter the energy exerted on the yoke up to the break will consequently be

$$a_o = Q_B e_B$$

If a residual stress P is imparted to the central bar so that it is elongated by the amount e, it will stretch over a shorter distance than in the former case, namely $e_B - e$ according to Figure 20 (middle right). The yoke can then withstand the load energy before the break:

$$a = Q_B (e_B - e) = a_o - Q_B e$$

In ordinary cases the energies a and a_o are very similar, since e can, at the most, assume the value of the material's yield point elongation e_s, and e_B is very much greater. Cases may occur, however, where e_B is small, as for example, in the case shown in Figure 20 (at the bottom). Here the elongation of the central bar is reduced by a notch such as a slag inclusion, a blow hole, an actual crack, or a notch as in impact testing, in which case the condition is still further accentuated by rapid loading and a lowering of the temperature. The relationship in these cases between the energies a and a_o is shown in Figure 20 (lower left).

According to Hooke's law we have

$$\frac{e}{e_s} = \frac{\sigma}{\sigma_s}$$

If we insert this in the above equation

$$a = a_o - Q_B e$$

we find that

$$a = a_o - Q_B e_s \frac{\sigma}{\sigma_s}$$

Here Q_B and e_s are dependent upon the material in question, the form of the notch and the speed and nature of the load. If the last three are constant (impact test on a V-notched Charpy bar), the product $Q_B e_s$ will be a material constant which we can denote by c, and we then find the following straight line

$$a = a_o - c\frac{\sigma}{\sigma_s}$$

As may be seen from the figures, the reduction of the energy is not caused by the residual stresses themselves but by the reduction in the remaining ductility, that is the exhaustion of the ductility which results from the residual stresses.

I shall now analyze an investigation made to confirm the said relationship. For that purpose Charpy test bars with a V-notch were used. Two different steels (A and B) were employed for the test. The analysis and strength of the steels we can see in Figure 21.

In both cases the material was normalized at 900° C. After the impact test bars had been completed they were stress annealed in an inert gas at 600° C.

The test bars were clamped between two U-shaped yokes as shown in Figure 22. By means of screws and threaded holes at each end of the bars, a tensile stress could be produced in the specimen. As may be seen from the figure, the bars were provided with gage marks on both sides of the notch at a distance of 9 mm apart. Similar marks were also placed on the opposite sides of the bars. The distance between the marks was measured by a sensitive tensometer while the screws were being drawn tight. The testing equipment was constructed in such a form that it could be conveniently placed in an ordinary Charpy impact testing machine. The yokes were slightly rounded at the ends so that they did not offer any appreciable resistance when they were bent under impact.

The results obtained from the impact tests are plotted in diagrammatic form in Figure 23 for steel B. This steel was tested with bars under different tensile stresses, and the stress along the bar is set out on the abscissa in terms of the yield point, σ_s. The impact energy is plotted on the ordinate. As may be seen, parallel straight lines are obtained as the relation between the impact energy and the ratio, σ/σ_s. The testing was conducted at temperatures of 0° and -30° C.

In Figure 24 the -30° C. straight line from Figure 23 is drawn and also the straight lines for steel A at -65° C. and -80° C. The equations for the lines are presented in the figure. As may be seen they closely confirm the theoretical considerations.

In Figure 25 the results of tests with steel A at different temperatures are plotted. A study of Figure 25 shows that as far as the influence of the residual stresses on the transition temperature is concerned, the determination of the energy equation is entirely dependent upon the manner in which the transition temperature is defined. Nevertheless, we see that the residual welding stresses raise the transition temperature.

The investigation shows that a brittle fracture will be initiated more readily in a region with positive residual stresses than in one which is free from stress. What this actually implies may be seen in Figure 26, which shows two similar regions (A and B) existing in a structure. In both regions there is a fault at x in the form of a blow-hole, a slag inclusion, a crack, or the like. The only difference between the regions A and B is that A is free from stress and B is subjected to residual stresses which are in equilibrium: the tensile stress σ_1 and the compressive stresses σ_2. The investigation has shown that a brittle fracture can be initiated more easily in the region B than in A, and it will start from the fault at x and will be propagated through the region under tensile stress and will eventually pass into the parts under compressive stresses. The temperature must be lower than the transition temperature.

PART II - METHODS FOR MEASURING RESIDUAL STRESSES

SURFACE STRESSES

When measuring residual welding stresses it is necessary to determine the local stresses as closely as possible and trace the peak values. For that purpose the measuring surface should have the smallest possible dimensions; in other words, the gage lengths should be short. This entails very small changes in length on relieving the stresses, and in order to obtain fairly exact values for the stresses, the changes in dimensions must be determined with a very small margin of error. This can be achieved by employing the following method.

Small depressions are made in the surface on the spot where the stresses are to be measured as shown in Figure 27. The diametric distances between these depressions are measured before and after the drilling of a groove round the measuring spot as shown by the broken lines in Figure 27. It is necessary to drill the groove to a depth of 8 mm. The surface of the central area is then free from stresses and the gage lengths are again measured. From the difference in the values between the two measurements, the stresses that have been relieved, and were therefore present originally, can be calculated.

The distances between the depressions are measured by employing a special tensometer with a very high degree of measuring accuracy. It is shown in Figure 28. The legs (1 and 2) are supported against each other by two points (3). The adjusting screw (4) is threaded into the leg (1) and the nut (5) fixed to the latter. The flat end of the adjusting screw lies against the gauge pin (7) of the accurate indicator (6). The indicator is fixed to the leg (2) by the holder (8). The parts (9) and (10) form a protective cover. This cover terminates at the bottom in a conical part (11). The adjusting screw (4) is accessible through a hole (12) for adjustment. A holder (13) with balls (14) is fixed to the lower part of the legs (1 and 2). When measuring stresses the balls rest in the depressions shown in Figure 27 and the distance between the depressions is read on the scale of the indicator (6).

SUBSURFACE STRESSES

The principle followed in the method of measurement of subsurface stresses is illustrated in Figure 29. The problem is to ascertain the magnitude of the residual stresses of a plate, both at the upper and lower surface of the plate and also inside the material in the parts between these surfaces.

Parallel holes are drilled through the plates. It has been found suitable to adopt a hole diameter of 3 mm and a distance of 9 mm between the holes. The distance between these two holes must now be measured with the greatest possible accuracy at a number of levels between $Z = 0$ and $Z = S$, where S is the plate thickness. It is preferable to select the following: $Z = 0$, $Z = 2$, $Z = 4$, $Z = 6$ mm, etc. After all these distances have been measured and recorded, the measuring point is freed from stresses by drilling a groove around it and the same measurements are repeated for $Z = 0$, $Z = 2$, etc., up to $Z = S$. The difference between the distances before and after relieving the stresses at each level will then provide a measure of the stress that had been relieved; that is to say, the stress existing at the level in question prior to its relief.

The stresses are relieved by freeing the measuring point from the surrounding material. A groove is milled along the dotted lines in Figure 29 around the measuring point, right through the plate, whereupon a cylinder detached from its surroundings is obtained (Figure 30). It must now be assumed that this cylinder is approximately free from stresses. This will be correct if the stresses at the respective levels did not originally vary diametrically through the cylinder. It is of importance, as when measuring surface stresses, to keep the dimensions of the cylinder as small as possible, i.e., approaching a mathematical point. Actually, one measures the stresses produced in the cylinder by the surroundings and these stresses are eliminated upon freeing the cylinder from its surroundings. Measuring biaxial stresses requires that four holes be drilled at the measuring point and the distance between two diametrically parallel holes at different levels in the measuring point be measured before and after the stresses have been relieved.

The direction of the diameters may suitably be located in the direction of the main stresses; for example, along and across the weld as shown in Figure 29.

The distances between these holes are now measured at different levels between the surfaces of the plate. For this purpose a device of the type shown in Figure 31 is placed on one side of the plate, preferably the lower side. This device consists of a tubular-shaped sleeve (1) and a pin (2) which can be displaced inside the former, and has a coarse screw thread (3) at its lower end arranged to slide in sleeve (4) which forms part of sleeve (1). The parts (1-4) cannot be turned in relation to one another on account of the guide pin (5) which is fixed in part (4). The upper part of pin (2) is conical and the sleeve (1) is here provided with one or more slots (6). Owing to this construction the sleeve (1) will expand if the pin (2) is displaced downwards in relation to the sleeve (1) A displacement of this kind can be produced by tightening the nut (7). The sleeve (1) and pin (2) are hereby locked against the walls of the hole at a certain level. The pin is provided at its upper end with a conical depression with very smooth surfaces. As will be seen, the unit shown in Figure 31 consists of two such devices which are combined with each other by a yoke (9) so disposed that the two parts can move freely in relation to one another but cannot turn. This movement is necessary since the parallelism and distance between the holes cannot always be maintained exactly the same.

The necessity for preventing pins (2), Figure 31, from turning in relation to one another and to the sleeves (1) is based on the fact that the conical holes (10) can never be constructed entirely concentric with the periphery of the pins (2). A rotation of the pins during the measurements after the relief of the stresses, in relation to their position during the first measurements, would invalidate the results.

The distance between the two conical depressions in the two pins (2) is measured by a special tensometer. The construction of this instrument is shown in Figure 32 and is described above.

TRIAXIAL STRESSES

In principle, the method consists of drilling four holes of 3-mm diameter and preferably 9 mm apart, right through the plate at the measuring spot according to Figure 33, which shows the holes in a weld viewed from above. The upper left sketch shows a section through the weld and holes. The holes should be truly parallel and reamed to size. A well-indicated mark is also made in the center between the holes at the top face and root side of the weld.

The distances L_a and L_b in Figure 33 are measured at the top face and root and at a number of levels between these surfaces at the distances $e_o = 0, e_1, e_2, e_3$, etc., from the top face of the weld. The dimensions $L'_{at}, L'_{a1}, L'_{a2}, L'_{a3}$, etc., in the longitudinal direction of the weld and $L'_{bt}, L'_{b1}, L'_{b2}, L'_{b3}$, etc., in the transverse direction are then obtained. The distance L_{eo} between the marks at the top face and root of the weld are also measured.

A groove is milled around the measuring spot as shown in the upper right of Figure 33 to a depth which is indicated as Z_1, and may suitably be taken as 2 mm. The distance between the marks at the top face and root is measured again and its length is now L_{e1}. The groove is then milled to Z_2 (Figure 33 middle left) which may be suitably taken as 4 mm. The distance between the surface marks will now be L_{e2}. The same procedure is followed until the groove has been milled through practically the whole of the weld metal and the distances L_{e1}, L_{e2}, L_{e3}, etc., are obtained. Next, the previously measured distance L_a and L_b between the holes are measured, and since the plug is free from its surroundings and therefore free from stress, these distances will now be (Figure 33 middle right) $L''_{at}, L''_{a1}, L''_{a2}, L''_{a3}$, etc., and $L''_{bt}, L''_{b1}, L''_{b2}, L''_{b3}$, etc.

We have now obtained sufficient measurements to be able to calculate the specific changes in length at different levels and denote these by ϵ. We get then the stresses by substituting in the equations of Figure 34.

The measurement of the dimensions L_a and L_b and the specific changes in length ϵ_a and ϵ_b have been described above. The measurement of L_e (values in the perpendicular direction) is carried out with an instrument which is illustrated schematically in Figure 35. A table is supported by three legs which rest in carefully prepared depressions on the surface of the plate. The legs can readily bend slightly. An indicator with a high degree of measuring accuracy is fixed on the table. A tongue is mounted below the table and a leg in this tongue is supported against a mark such as a conical depression at the center of the measuring spot. This depression guides the instrument in a lateral direction. A pin in the tongue actuates the indicator's measuring point. By means of the leg which is threaded into the table, the indicator can be set at zero. This screw must not be moved subsequently while the measurements are in progress.

By means of this arrangement, therefore, the height of the measuring leg relative to the other legs can be read off, and consequently the height of the measuring surface also in relation to the fixed points on which the legs rest. This reading is taken for every Z-value as described above. Since it is not possible to exclude a change in the root surface due to the milling of the groove in the top face, measurements are taken on the root surface in an identical manner after the plate has been turned.

In order to compensate for any differences in temperature, etc., the actual distances are never measured. Instead, one measures how much each distance differs from a given fixed distance, a reference distance.

Instead of one mark in each top face and root surface, a number may be employed and, for greater accuracy, the mean value of these may be taken.

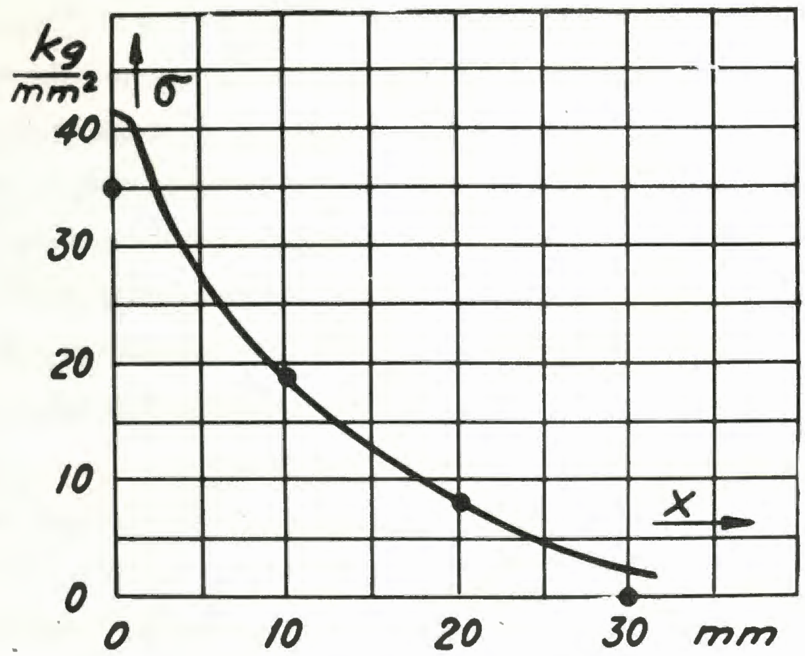


Figure 1 Transverse residual stresses in longitudinal weld of 3600 mm casing.

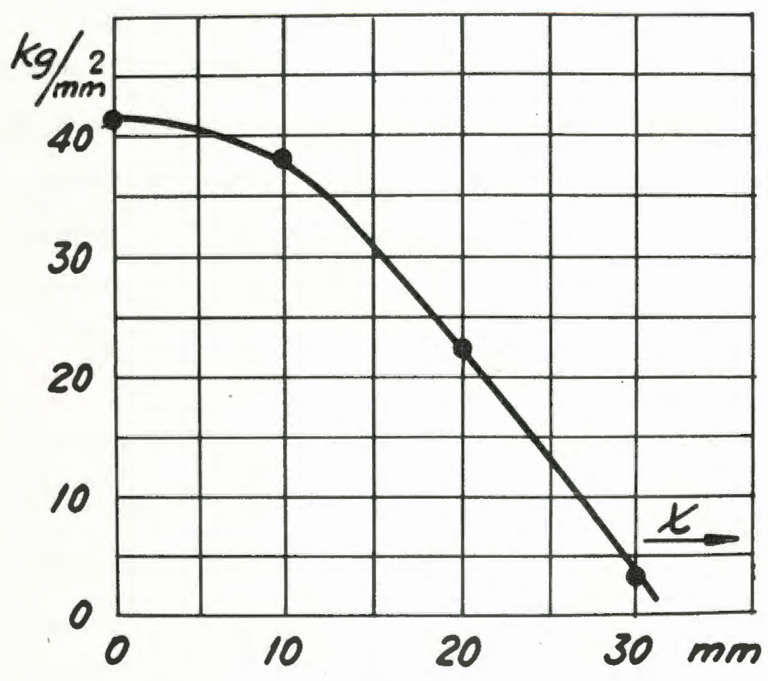


Figure 2 Longitudinal residual stresses in longitudinal weld of 3600 mm casing.

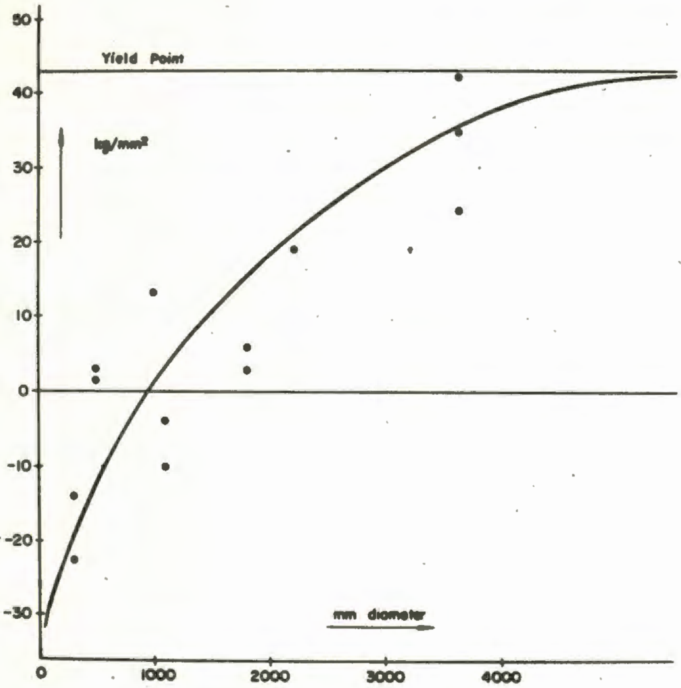


Figure 3 Transverse residual stresses in longitudinal welds of various size vessels.

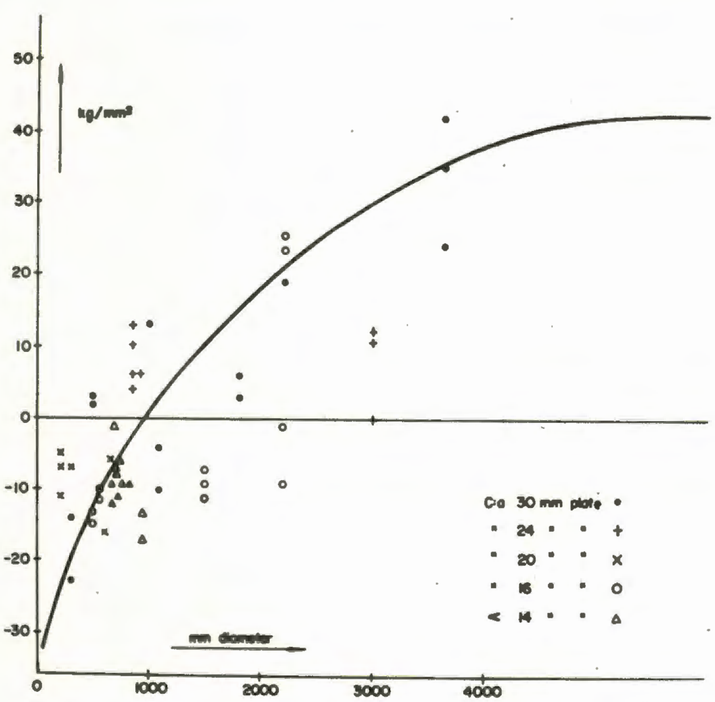


Figure 4 Transverse residual stresses in longitudinal welds of vessels fabricated of various thicknesses of plate.

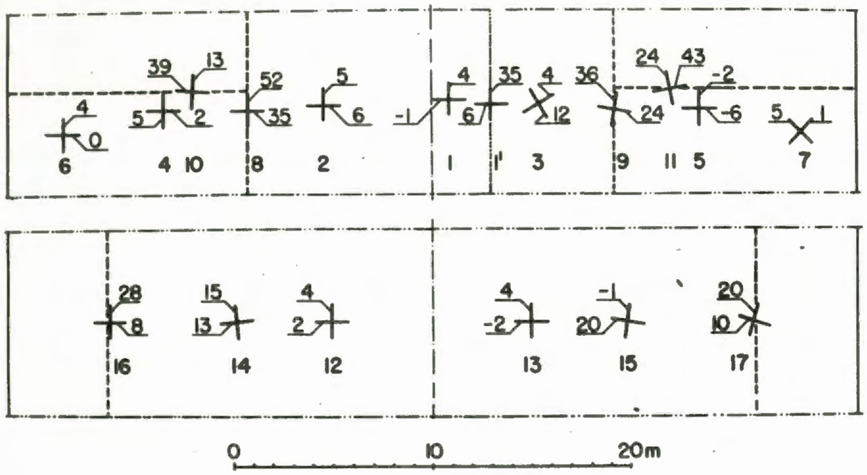


Figure 5   Residual welding stresses in hull of tanker prior to launching.

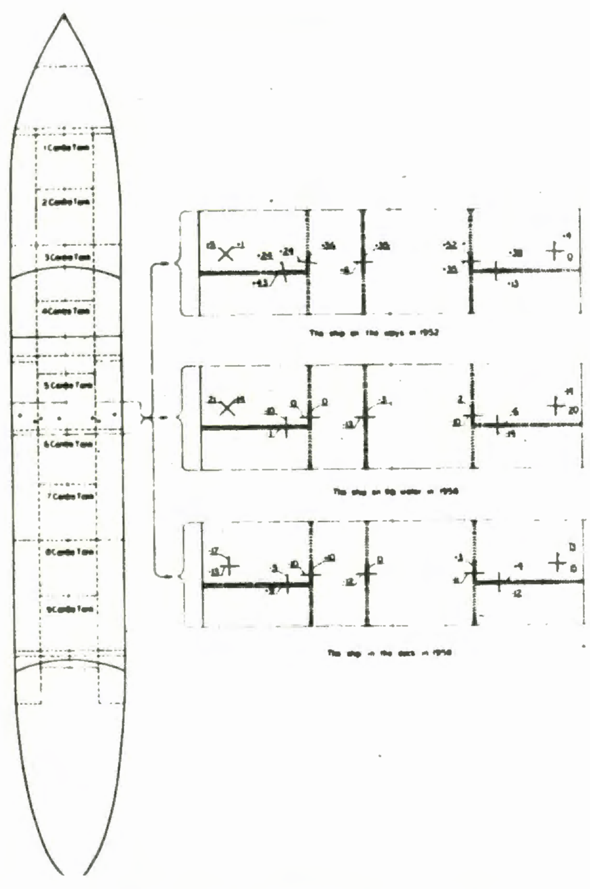


Figure 6   Residual welding stresses in deck of tanker prior to launching and after six years of service.

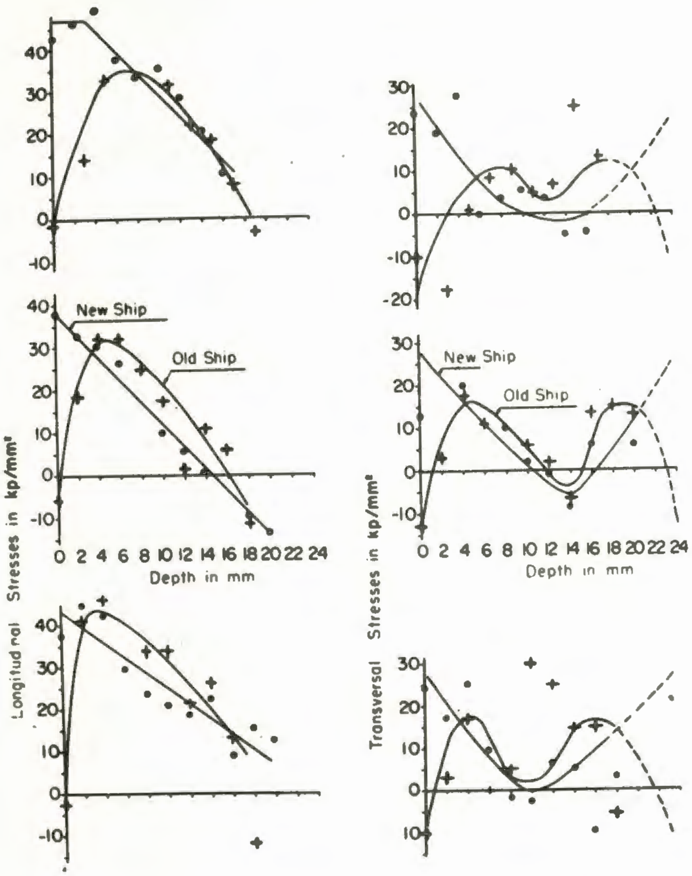


Figure 7 Effect of service on residual stresses in M/T tankers.

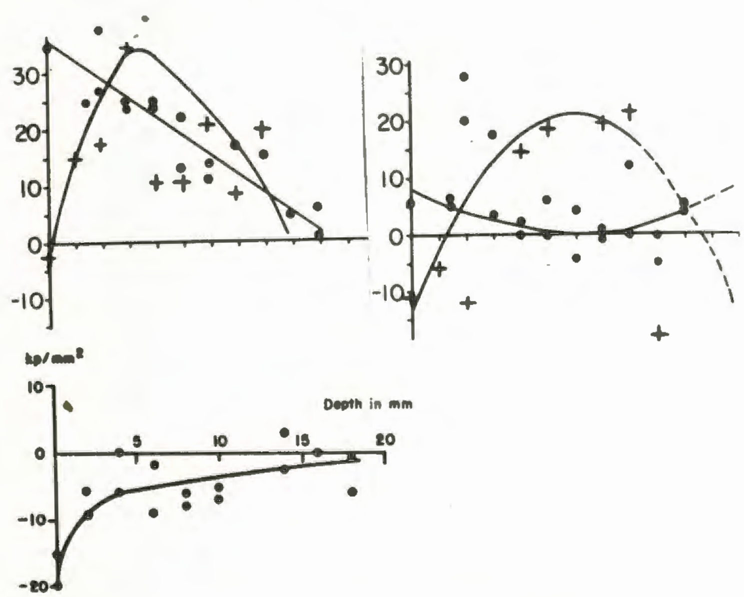


Figure 8 Residual stresses in submerged arc welds.

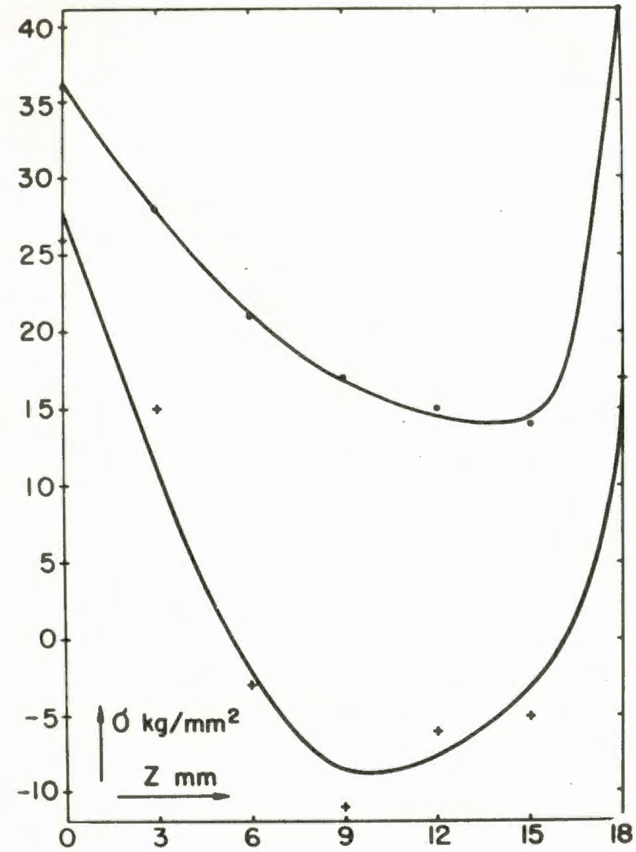


Figure 9 Stresses in multi-pass welds.

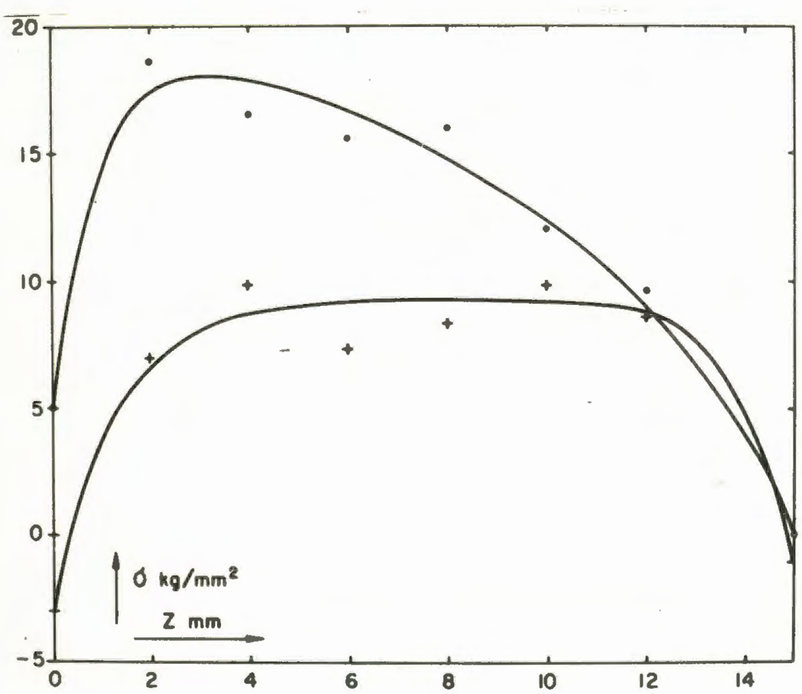


Figure 10 Residual stresses in single-pass submerged arc weld.

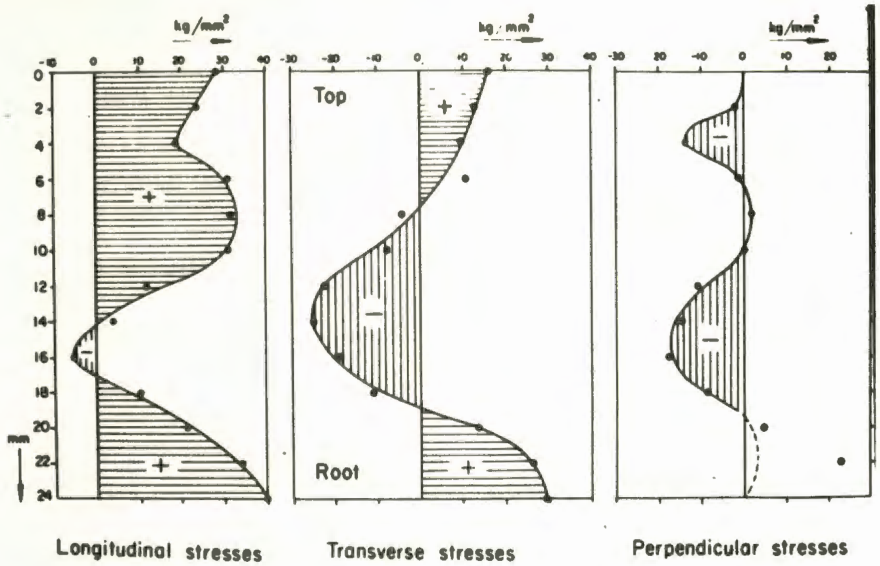


Figure 11 Triaxial residual stresses.

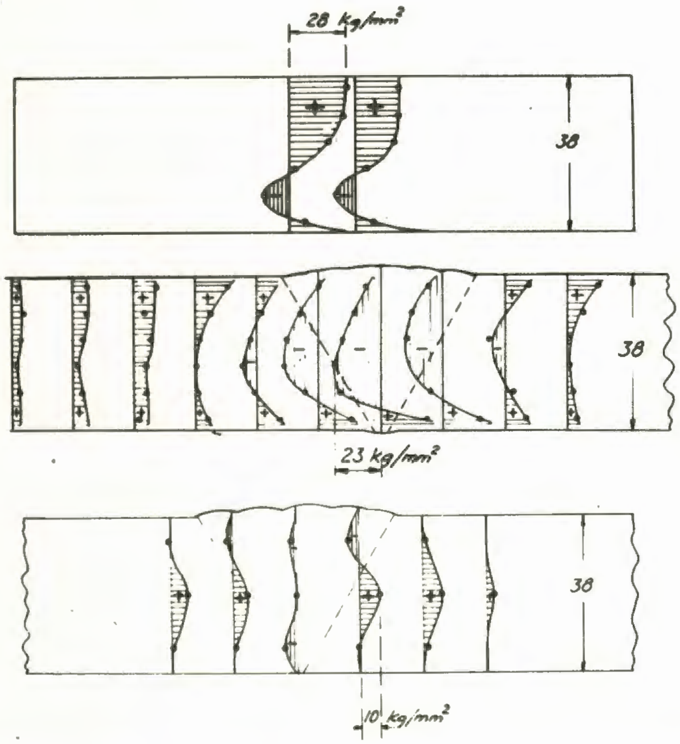


Figure 12 Longitudinal, transverse and perpendicular stresses in welded joint.

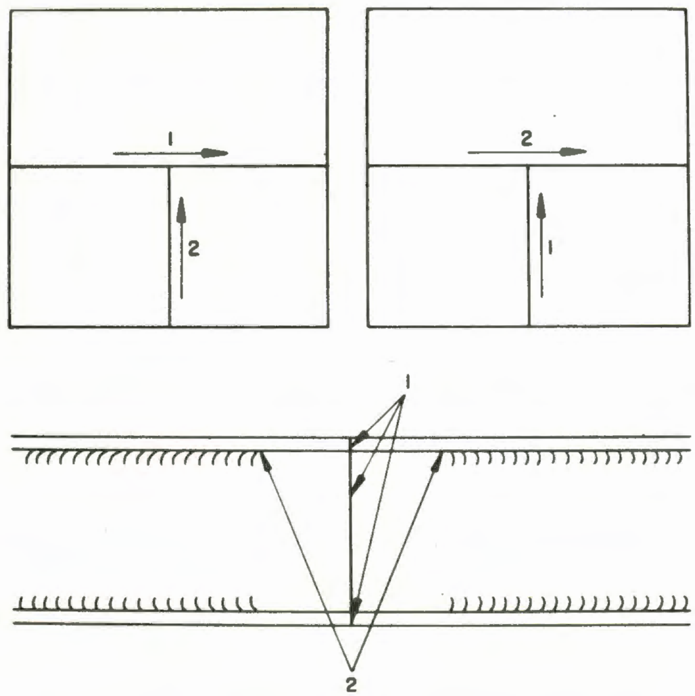


Figure 13 Sequences for intersecting welds.

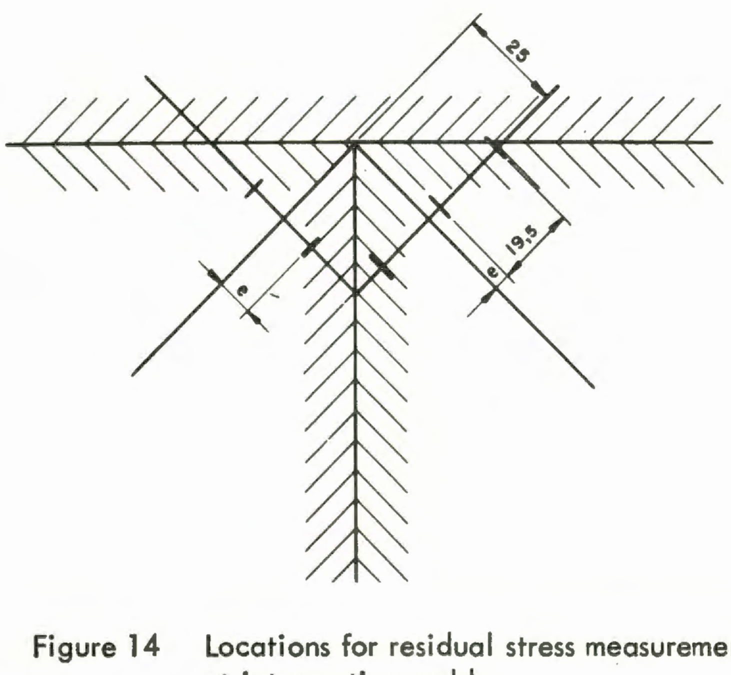


Figure 14 Locations for residual stress measurements at intersecting welds.

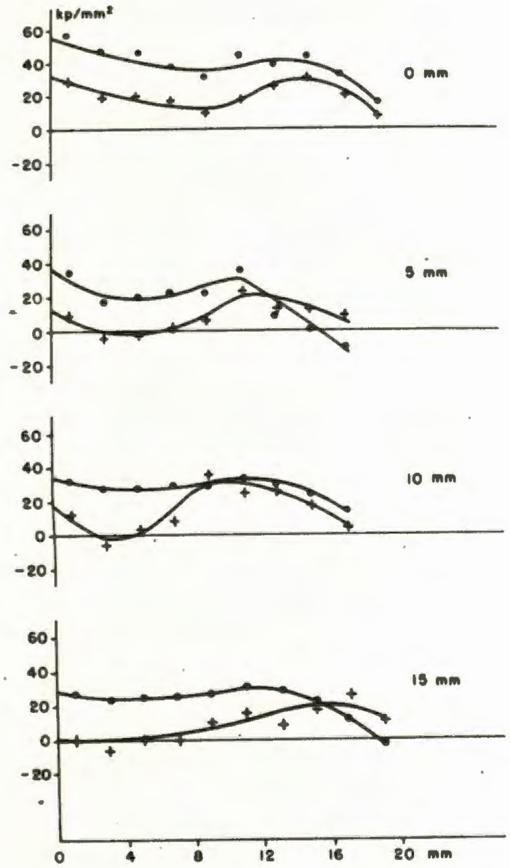


Figure 15
Residual stresses at intersecting welds.

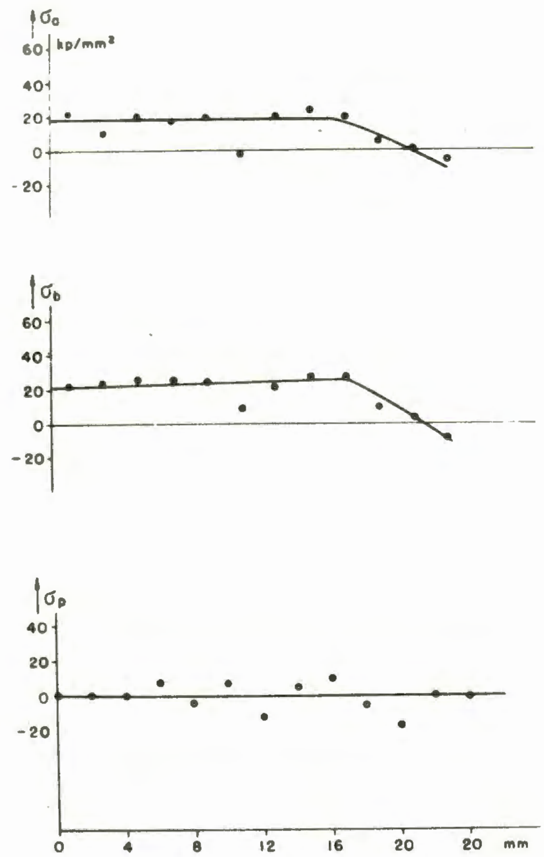


Figure 16
Longitudinal, transverse and perpendicular stresses at intersecting welds.

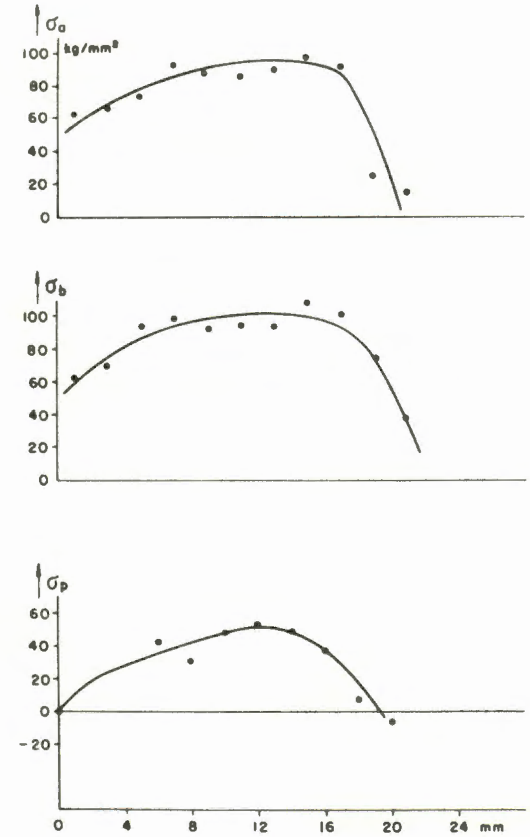


Figure 17 Unusually high longitudinal, transverse, and perpendicular stresses at intersecting welds of one member.

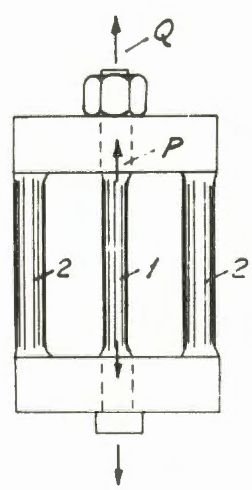


Figure 18 Simplified model for analysis of weld stresses.

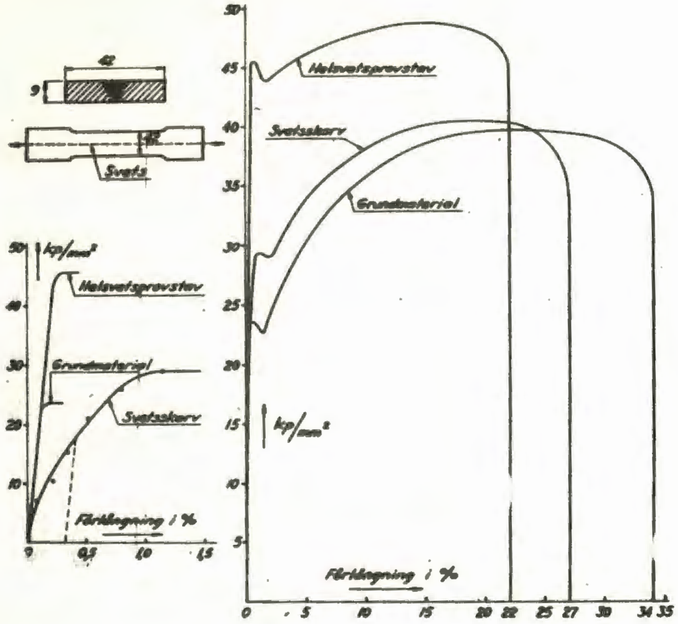


Figure 19 Effect of weld on stress-strain properties of a weldment.

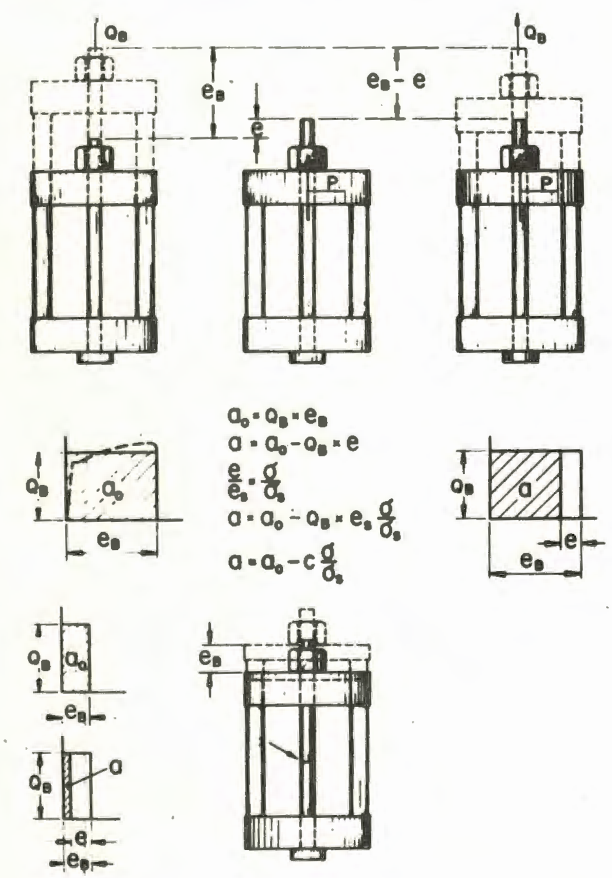


Figure 20 Model to demonstrate energy absorption at failure.

		Steel A	Steel B
Analyse:	C	0,18 %	0,19 %
	Si	0,26 "	0,26 "
	Mn	0,73 "	0,76 "
	S	0,011 "	0,03 "
	P	0,005 "	0,03 "
	Al	0,038 "	-
	N	0,007 "	-
Yield point σ_s =		36 kg/mm²	26 kg/mm²
Tensile strength		51 "	44 "
Elongation		25 %	20 %

Figure 21 Properties of steels A and B.

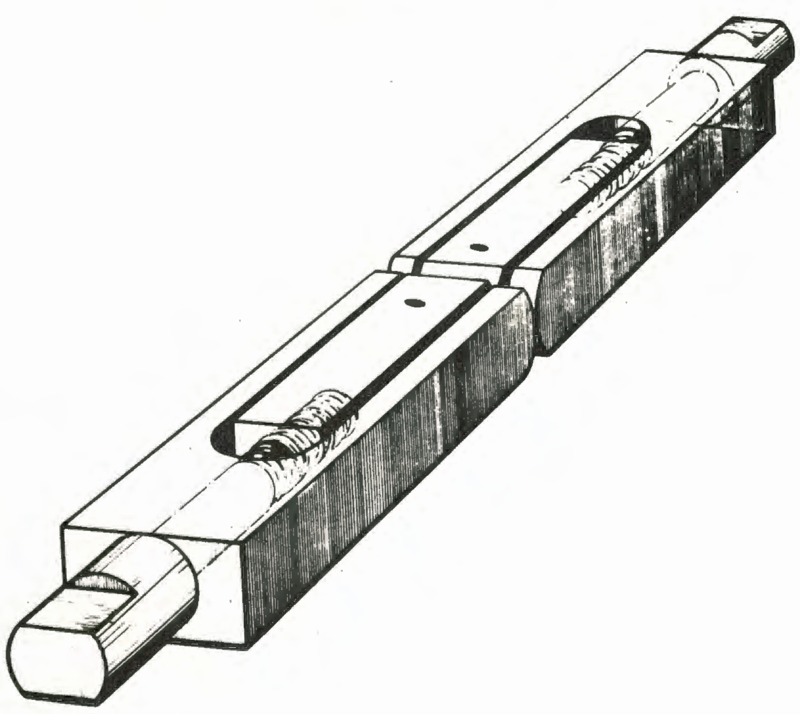


Figure 22 Test fixtures for tests of Charpy specimens.

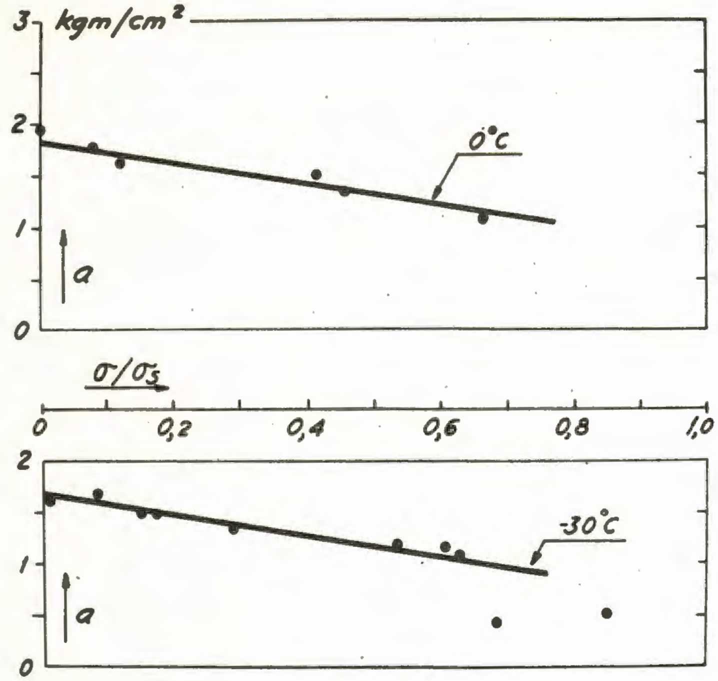


Figure 23 Energy absorption of Charpy specimens subjected to axial tension.

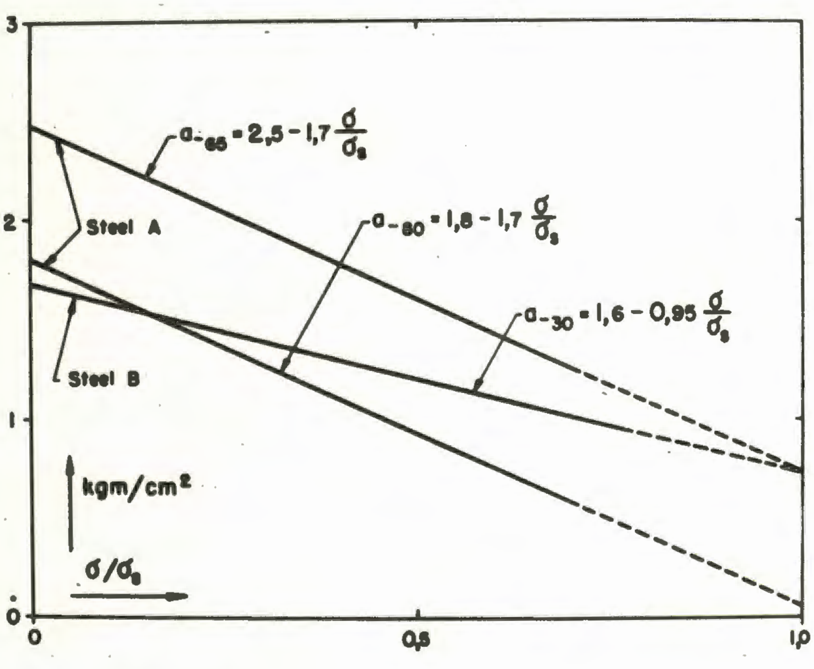


Figure 24 Relationships between impact energy and residual stress for two steels tested at several different temperatures.

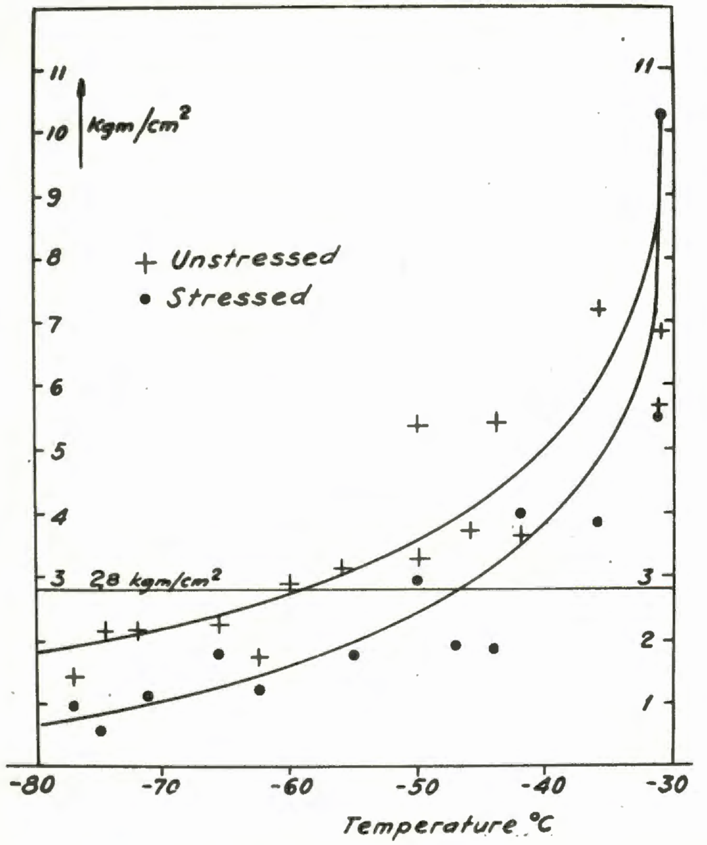


Figure 25 Effect of residual stress on energy absorption of Charpy specimen.

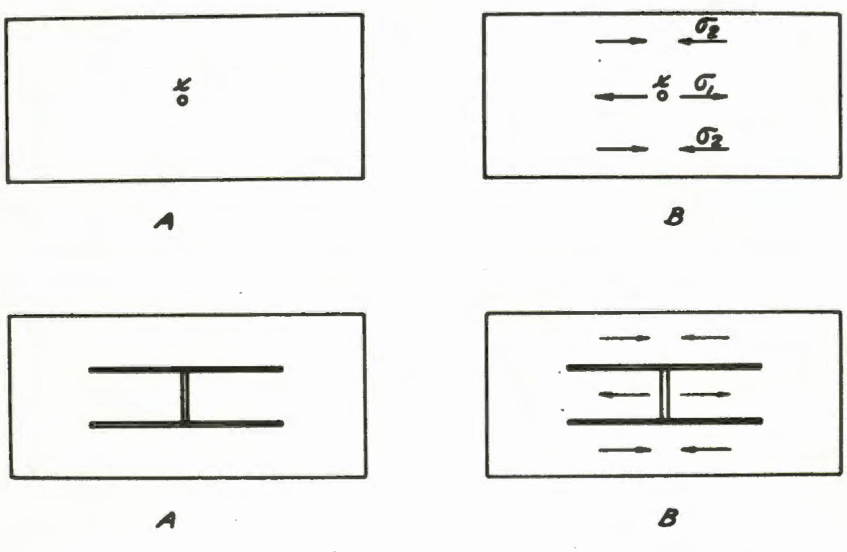


Figure 26 Members with flaws to demonstrate the effect of residual stresses on brittle fracture.

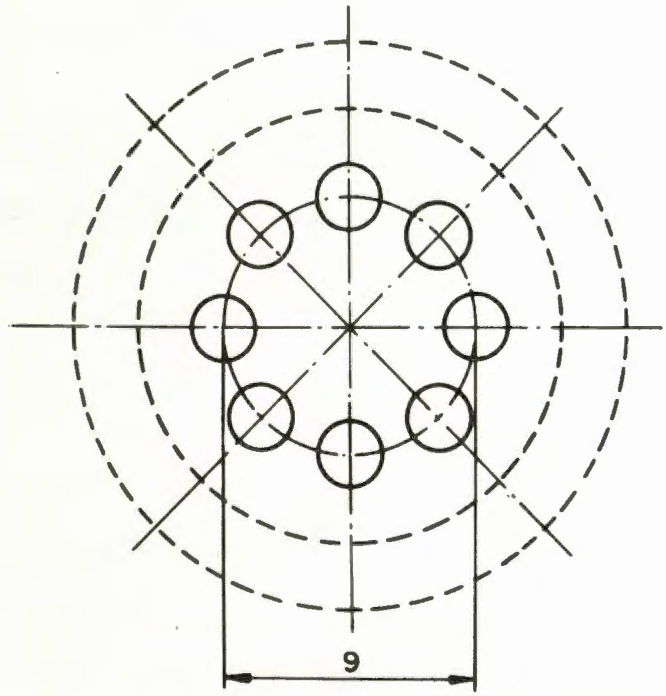


Figure 27 Gage hole locations for residual stress measurements.

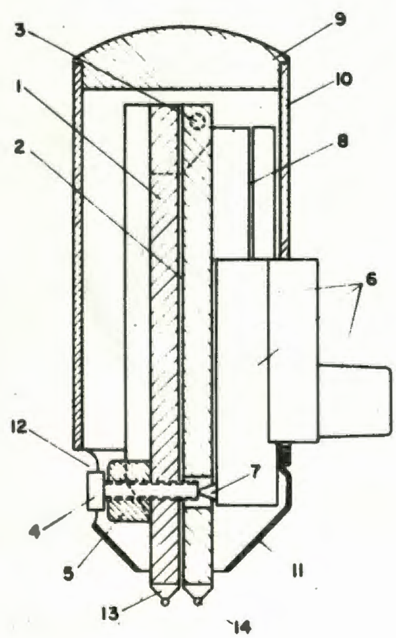


Figure 28 Special high-accuracy tensometer.

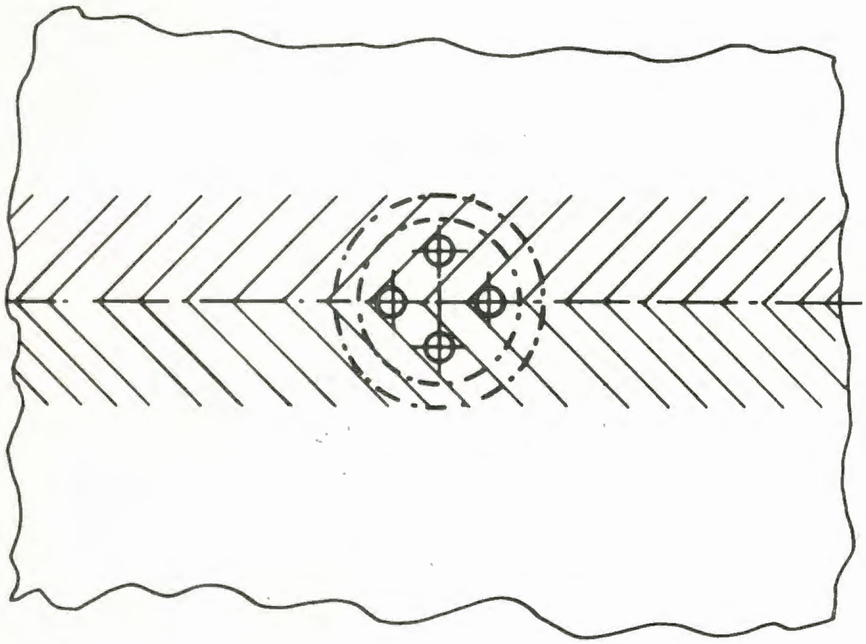


Figure 29 Gage holes in weld metal for residual stress measurements.

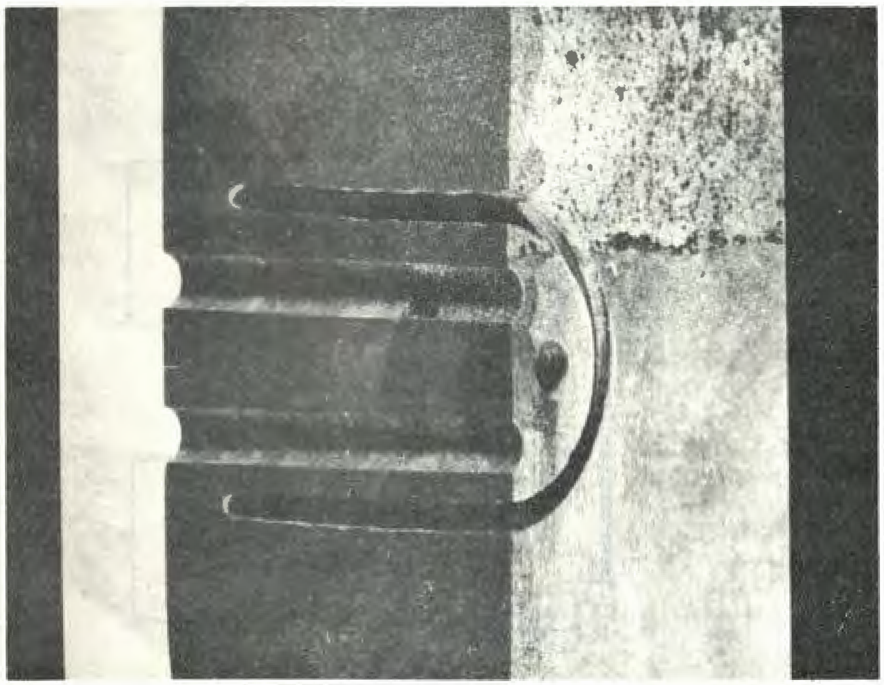


Figure 30 Gage holes and milled groove at the measuring point.

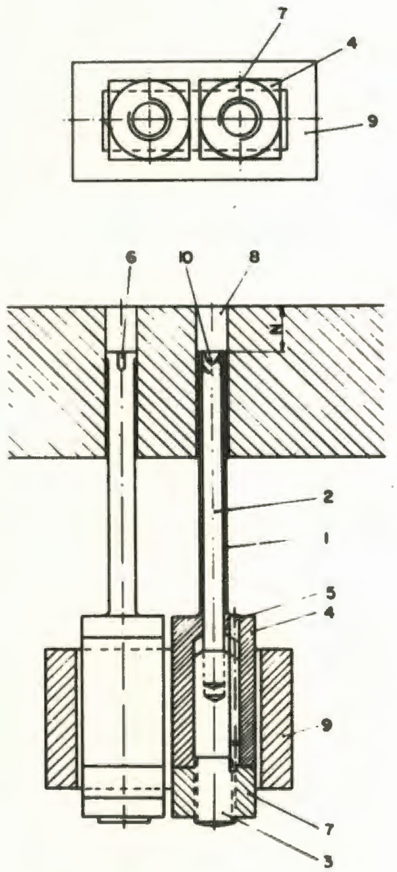


Figure 31

Support for the legs of the tensometer in order to determine the residual stresses through plate thickness.

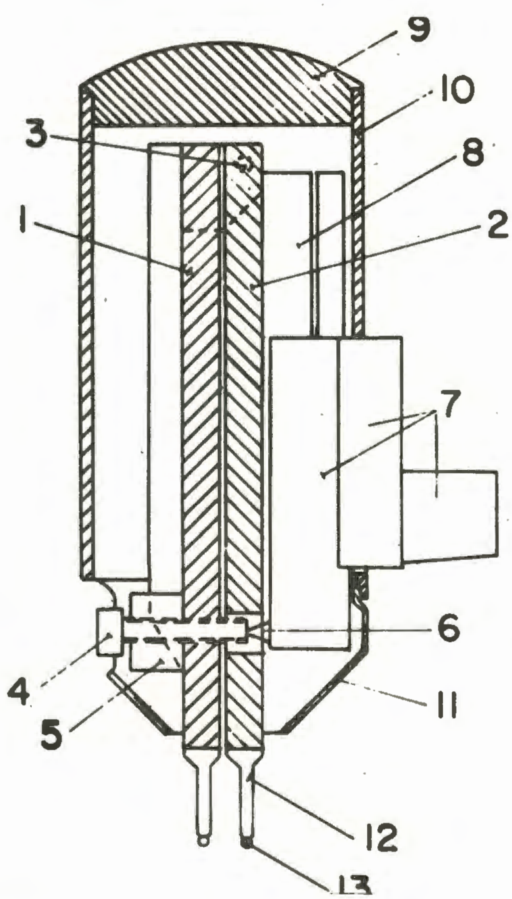


Figure 32

Special tensometer for measurement of residual stresses through plate thickness.

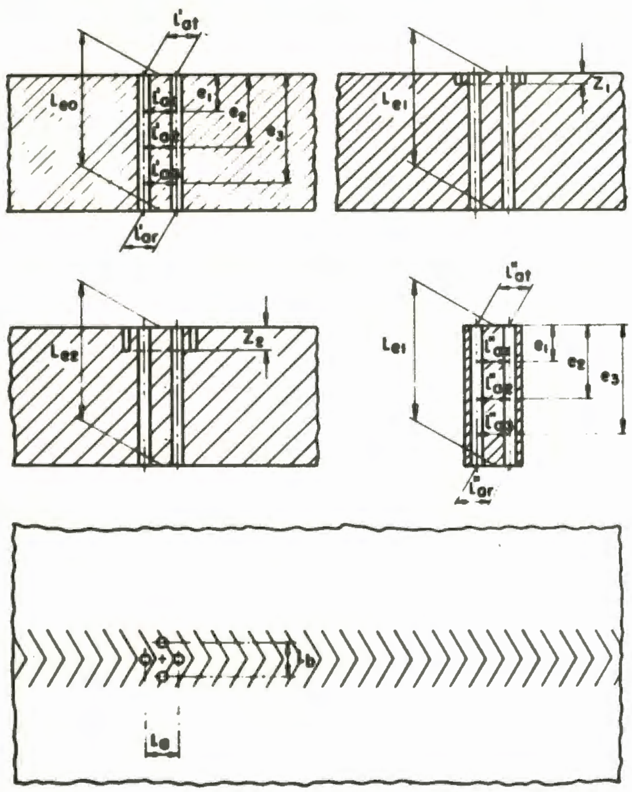


Figure 33 Determination of triaxial residual stresses in a weld.

$$\sigma_a = 27\,\varepsilon_a + 11{,}5\,(\varepsilon_b + \varepsilon_e)$$

$$\sigma_b = 27\,\varepsilon_b + 11{,}5\,(\varepsilon_a + \varepsilon_e)$$

$$\sigma_e = 27\,\varepsilon_e + 11{,}5\,(\varepsilon_a + \varepsilon_b)$$

Figure 34 Equations to clarify triaxial stresses by procedure of Figure 33.

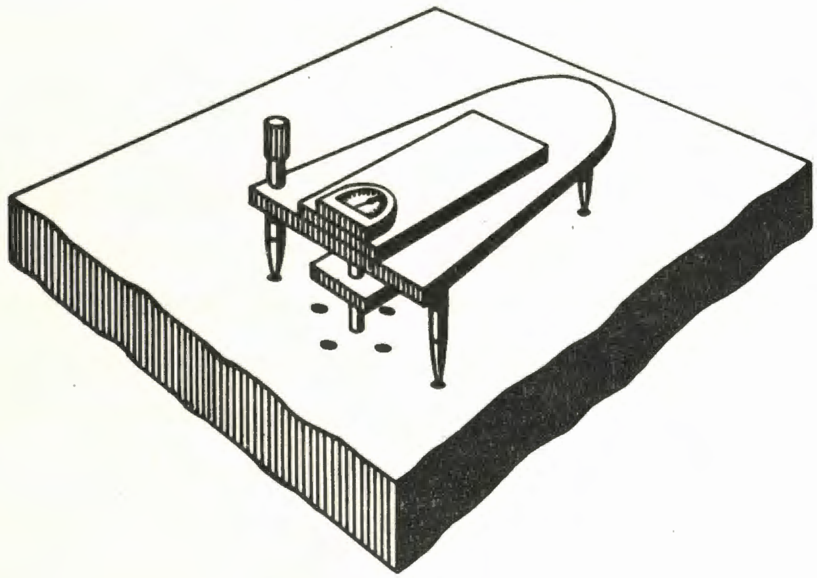


Figure 35 Instrument for measurement of perpendicular residual stresses.

SELECTION OF MATERIALS AND CONSTRUCTION TO AVOID FAILURES

Karl Ruehl, Professor, Vice-president of the Federal Laboratories for Testing Materials, Berlin-Dahlem, Germany

We know that failures in welded structures may occur because of an embrittlement of the material. In fact, brittle fractures have proven to be the cause of grave accidents, of failures of ships, bridges, pipe lines, storage tanks, and other constructions.

In many interesting papers it has been shown that a number of factors have an important influence on the brittle behaviour of structural parts, and that rather complicated correlations exist between the embrittlement and the intrinsic properties of the steels, the rate of loading, the thickness of the plates, the residual stresses, etc. Much essential research work has been carried out in this field, but the decisive problems are far from being solved.

The practicing engineer cannot be satisfied by detailed results of investigations into single problems, however interesting they may be. Neither can he wait till all the problems are solved; he has to build his structures now and he has to choose an appropriate steel for his structural parts. He needs, therefore, an understanding of the behaviour of steels in order to foresee the behaviour of the steel in his structures and to guarantee the necessary safety in these structures. He needs not only a description of the design requirements, but he needs also recommendations for the choice of steels and summarizing regulations or rules, however fragmentary or incomplete they may be. For this purpose we will consider three aspects.

(1) The basic aspect of the problems and the work completed to solve these basic problems.

(2) The tests carried out to gain results which enable the engineer to judge the danger of brittle fracture in practice.

(3) The regulations which have been established to prevent brittle fractures.

BASIC ASPECTS

A study of the failures and of the great amount of scientific work carried out in the U.S.A., and in other countries as well, has shown that the following philosophy seems to give a good explanation for the problem. The ductility of steels depends on three factors:

(1) The inherent properties of the steel, the chemical composition, the grain size, the microstructure.

(2) The temperature.

(3) The stress conditions, as Mr. Lagasse explained, i.e., the state of stress and the rate of strain.

A combination of these factors results in the following essential conclusion: Every steel looses its ductility in all specimens below a certain temperature, but this critical temperature (transition temperature) is not a constant value for the material; it depends on the triaxiality of stresses and the rate of strain. Therefore, we cannot say that a particular steel has a certain transition temperature; we can only say that a steel has a certain transition temperature under certain conditions in a given specimen. It then follows that the transition temperature of a steel used in a structural part, for instance in a welded joint of a bridge, differs from the transition temperature of the same steel as found in a Charpy-V or other type of specimen. That is the basic aspect of the problem but many other questions arise. The first concerns the correlation or the interdependence between these three factors. In the theoretical field we have to consider the influence exercised by rate of strain and the triaxiality of stress. We can use the fundamental correlations of the embrittling factors for a practical estimate only if the numerical correlations are known as well. As regards the strain rate, we know that there exists a correlation between strain rate and critical temperature. If we use a system of coordinate axes in which the x-coordinates (abscissa) are the reciprocal values of the absolute temperature, and the y-coordinates (ordinates) are the strain rate in a logarithmic scale, the corresponding values of x and y form a straight line which separates the ductile and brittle range (Figure 1). The straight line may be represented by the equation shown in the figure. The values C_1 and C_2 in this equation have been examined in many tests. Thus the correlation is known to a sufficient degree.

However, no general criterion has been known till now concerning the influence of triaxiality. We know of course of the diminution of the plastic deformation in the case of triaxial stresses as compared with the deformation under a uniaxial stress. But we do not know the magnitude of the shift in critical temperature actually associated with the decrease of deformation.

Tests carried out at the Max Planck Institute for Iron Research, Düsseldorf, by Prof. Kochendörfer and Mr. Scholl, give a survey and some interesting results. Prof. Kochendörfer has used a quantity already used in a similar way, but for other purposes, by Prof. Baes. Professor Kochendörfer bases his philosophy on the well-known distortion-energy criterion for yielding under multiaxial stresses.

This criterion gives:

$$0.707 \sqrt{(\sigma_1-\sigma_2)^2 + (\sigma_2-\sigma_3)^2 + (\sigma_3-\sigma_1)^2} = \sigma_y$$

where

$$\sigma_1, \sigma_2, \sigma_3 \doteq \text{principal stresses}$$

$$\sigma_y \doteq \text{yield point.}$$

For a uniaxial state of stress, $\sigma_2 = \sigma_3 =$ zero and yielding occurs when $\sigma_1 = \sigma_y$. For a triaxial state of stress, $\sigma_2 > 0$, $\sigma_3 > 0$ and yielding can only occur if the maximum principal stress σ_1 is greater than σ_y. For a complete triaxial state of stress, in which $\sigma_2 = \sigma_3 = \sigma_1$, we would obtain $\sigma_y = \infty$. In reality, of course, the cleavage strength will be surpassed and a cleavage fracture will occur without previous yielding.

If we now use the quantity $k = 1 - \sigma_y/\sigma_1$ we obtain,

$k = 0$ for a uniaxial state of stress $(\sigma_2 = \sigma_3 = 0)$

$1 > k > 0$ for a triaxial state of stress $(\sigma_1 > \sigma_2 > \sigma_3 > 0)$

$k = 1$ for a complete triaxial state of stress $(\sigma_1 = \sigma_2 = \sigma_3)$

The value of $k = 1 - \sigma_y/\sigma_1$ increases therefore as the triaxiality increases. Thus, k can be used as a measure of the triaxiality or as a constraint factor. (Of course this value differs from the often used constraint factor $[(\sigma_1/\sigma_1) + (\sigma_2/\sigma_1) + (\sigma_3/\sigma_1)]$ Professor Kochendörfer has now for several steels examined four cases.

(1) He has determined the yield point of unnotched tension bars, σ_y.

(2) He has examined the yield strength of notched tension and flexural bars, σ_1.

He has thereby introduced the value of the nominal stress in the case where the whole or nearly the whole section has undergone plastic deformation.

(3) He assessed the fracture strength of unnotched bars.

(4) He assessed the fracture strength of notched bars.

The values have been verified for a wide range of temperature.

An example of the findings for one form of specimen is given in Figure 2. You may note that we have, for this particular specimen, an intersection of the curves for fracture strength and yield stress. Therefore, a brittle behaviour of the steel occurs at -160° F. Such curves are well known and many similar examples have been shown for example by Prof. Cohen.

Professor Kochendörfer has also plotted the critical temperature for several specimens and the coefficient, k, of these specimens. Then we obtain the relationship between critical temperature and the factor, k, as shown in Figure 3. In this way we obtain a criterion for the correlation between triaxiality and transition temperature. However, it should be understood that these tests cannot today resolve the problem as a whole. They are only a beginning and have to be continued.

CONSIDERATION OF PRACTICAL METHODS FOR JUDGING THE DANGER OF BRITTLE FRACTURE IN STRUCTURAL PARTS

The results hitherto shown are important in order to elucidate the basic problem. Unfortunately, they are not adequate to judge the danger of brittle fracture in practice, because we do not know the actual stresses in such parts.

It is very difficult to determine these stresses because residual stresses are added to the stresses caused by the loads, the dead weight, etc. There has been much research work concerning residual stresses, for instance, by Mr. Gunnert and Prof. Soete. There also has been much discussion as to whether residual stresses affect the sensibility to brittle fractures. Some investigations show no influence, but recent special tests by Prof. Kihara in Japan on wide plates have shown a large decrease of the transition temperature in plates with residual stresses when compared with stress-relieved plates. I feel that this divergence may be explained by the fact that residual stresses are very different in some cases. Biaxial residual stresses may have no unfavourable influence, but embrittlement will appear if triaxial residual stresses exist.

This uncertainty compels the practicing engineer to seek a way of judging the danger of brittle fractures without a complete knowledge of all theoretical aspects of the problem, and to consider what can be done for a practical suggestion. We think we can approach this problem in the following way.

Each member has a critical temperature, above which no brittle fracture appears. The risk is imminent only below a certain temperature. Of course we cannot make tests on structural parts with all grades of steel. But we hope that we can apply the procedure, indicated by the following proposal.

We determine (Figure 4) the transition temperature T_A of a particular steel, for instance steel A, and a particular type of test specimen, perhaps a Charpy specimen. Then we ascertain the critical temperature ($T_{structure}$) of the structural parts or semi-practical parts; that is, parts which are similar to the actual members. Such parts are used because full scale tests are seldom possible. Since the state of stress, the dimensions, and the strain rate in such structural parts are different from those in the specimens, we obtain a shift of temperature, ΔT. Of course the magnitude of ΔT depends on the design of the structural parts, the thickness, etc. The value of ΔT found in a test is only valid for structural parts with a similar shape and similar conditions of loading.

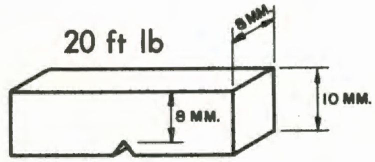
Now we have to make an assumption. If we know the value of ΔT for one steel, we may assume that this value is valid for other steels as well. Then, if we know the test specimen transition temperature T_B of a second steel B, we may evaluate the critical temperature $T_{Structure}$ for structural parts in steel B by adding the value ΔT to T_B. If then the service temperature at which the structure has to work, is higher than this calculated critical temperature $T_B + \Delta T$, no brittle fractures are to be feared. This method thereby gives a possibility of judging whether a particular steel is adequate for use in a particular structure. If, conversely, we know the lowest possible service temperature of a structure and the value of ΔT, we may indicate the highest allowable transition temperature of the steel in the test specimen.

If we want to apply this method, several conditions should be satisfied. First, the shift of temperature between the test specimen and a particular member must be the same for all the steels. Till now, we have not had enough tests to check this supposition. However, we have many results from test specimens and may compare, for instance, the Charpy-keyhole-notch tests with wide notched plates as examined in U.S.A.

Figure 5 shows that a number of the steels studied had approximately the same values of ΔT. I must add that we have other tests with similar results, but also different tests in which the correlation is less satisfactory. This seems to be a problem which must be solved and in which the correlation may be either good or bad.

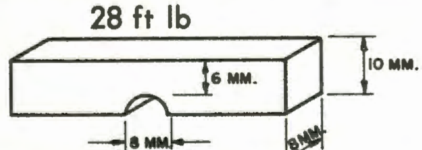
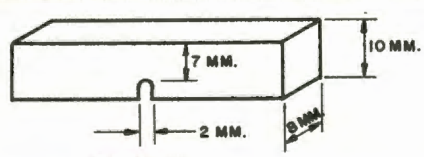
The second requirement for an application of this method is a knowledge of the transition temperature of the steels in the test specimens. This knowledge is, for many cases, ensured by the standards or specifications. For instance, the British Standard 2762 prescribes that the four standardized grades of steel are characterized by different temperatures at which an energy-absorption of 20 ft lb in the Charpy V notch test is required.

British Standard 2762 for Structural Steels

Steel	Criterion	Temperature °C.	Temperature °F.
ND I		0°	32°
ND II	20 ft lb	-15°	5°
ND III		-30°	-22°
ND IV	Charpy-V-notch	-50°	-58°

The German Standard DIN 17 100 uses a more complicated system. It requires different values of energy absorption for different test specimens (keyhole specimens, and bars with flat circular notches of 8-mm diameter) for different temperatures.

German Standard DIN 17 100

Steel	Energy Absorption	Temperature °C.	Temperature °F.
1	-	-	-
2	28 ft lb  aged condition	+20°	+68°
3	 35 ft lb	0°	+32°

But a large number of tests have shown that these requirements result in different and graduated values of transition temperature as well.

If we use a keyhole specimen, according to the German standard (DVM-specimen, DVM = Deutscher Verband für Materialprüfung) and a critical value of energy absorption of 18 ft-lb, the transition temperature for steel no. 2 would be equal to about 15° C. (for a thickness of 0.8 in), and -5° C. for steel no. 3.

A third requirement for the application of the ΔT method is a knowledge of the shift of temperatures between the critical temperatures of the test specimens and the structural parts. To obtain such information the German Committee for Structural Steelwork has sponsored tests of welded beams. Perhaps it should be mentioned that in Germany the development of this problem is somewhat different than in most other countries. In the U.S.A., the United Kingdom, Japan and in many other countries, the shipbuilding industry was at first most concerned with the problems of brittle fracture. In Germany we had no such failures in ships but, before World War II and before the casualties with Tankers and Liberty ships, we had two not dangerous but very interesting failures in welded bridges. Consequently, before World War II our steel building firms and the scientists working with structural steels took the lead in this field. That is one of the reasons that today our committee for structural steel work is interested in this subject. A second important factor is the relationship between labor and steel costs. If we compare the prices of the material and the costs of man-power, this proportion is different in Germany and in Europe as a whole than in the U.S.A. The steel is, compared with the wages, much more expensive in Europe than in the U.S.A.; and, on account of this fact, it is much more important for the European countries, both to save steel by a well designed and exactly calculated construction, and to avoid the application of a better steel than necessary. If we apply a higher grade of steel the costs increase. This is especially important for the steel-structures industry that fabricates many different types of structures: large buildings, small buildings, simple constructions, very complicated constructions, plates of large thickness, thin plates, etc. Therefore, the severity of the conditions to which the members are subjected varies greatly and the range of steel grades for a given set of conditions is rather large.

Because of this situation the structural steel industry is very anxious to find reliable and simple methods to choose the appropriate steel for the various structures.

We have manufactured welded test beams. These beams have been loaded by transverse loads up to fracture at different temperatures, and demonstrate that at low temperatures the beams fractured at small deformations. (Figure 6) The deformations, not the loads, decreased in a regular manner as the temperature decreased.

Now we can see that a dangerous decrease of ductility does not happen for temperatures above a certain value, in this case above a temperature of -60° F. Furthermore, we can assess the difference of temperature between the critical temperature of the welded beam and the Charpy transition temperature of the steel. The Charpy specimen was the keyhole specimen of the German Society for Testing Materials DVM. This transition temperature was nearly 0° F. and therefore in this case the shift of transition temperature was nearly 60° F. As we had expected, this difference proved to be dependent on the dimension of the structural part. It was found that the critical temperature of the welded beams increased, if the thickness and the width of the flanges were increased. Unfortunately, tests on such members are not simple and are rather expensive. Hence, until now it was possible to carry out only a small number of tests. The tests will be continued, but it will take time to extend the tests in the desired manner.

REGULATIONS FOR THE CHOICE OF STEELS

Since it has not been possible till now to carry out enough tests on structural parts, the steel work industry has been forced to consider the different standardized steel grades currently available and to choose the one adequate for each building. We believed it necessary to develop recommendations for classifying structural parts with respect to their susceptibility to brittle fracture. The German

Committee for Structural Steel Work has prepared such recommendations (Vorläufige Empfehlungen zur Wahl der Stahlgütegruppen für geschweiBte Stahlbauten, 2. Auflage; -- Preliminary recommendations for the choice of the steel quality of steels to be used in welded constructions, 2nd edition). The first suggestions have been submitted by Prof. Klöppel of the Institute of Technology at Darmstadt. Further suggestions have been forwarded by Prof. Bierett. The difference between the two systems is not very large.

The basic concept of the "Recommendations" is as follows. The severity of the conditions to which the structure is subjected is specified numerically. The danger-value takes into consideration all important factors. Points are allotted to the structure on the basis of the factors which influence the liability to brittle fracture. For severe conditions the danger-value is high. On the other hand the quality of the steels has to be characterized by a numerical value as well. The structure will prove to be safe if the quality value of the steel is equal to or higher than the danger value for the structures.

The factors that are taken into account are the following:

Factor 1:

The position of the weld seams with respect to the overall stresses: If a longitudinal weld bead is situated at a highly stressed part of a girder, for instance at the flanges, the factor is assumed to be 4-5. If the stresses are not high, perhaps near the neutral axis of beam, the factor may be diminished. For welds under biaxial stresses, for instance in pressure vessels or pipe lines, the value is 5.

Factor 2:

The character of the stress, whether the stress results from a permanent load (dead load) or from a temporary load (live load): If the share of stress arising from permanent load amounts to more than 50 per cent of the total stress the factor shall be 1, if it is more than 66 per cent of the total stress the factor shall be 2, and if it is more than 80 per cent of the total stress the factor shall be 3.

Factor 3:

The restraint developed in manufacturing: If the shrinkage arising from the welding procedure is prevented by very rigid construction, we shall have high residual stresses and in this case the factor is high. This factor shall have a maximum of 5.

Factor 4:

Thickness: For thickness less than 14 mm or 0.55 in., the factor shall be zero. For each 5 mm or 0.2 in. increase, the factor shall be enhanced by one. The maximum value shall be 5.

Factor 5:

Straightening: If work hardening by a straightening procedure occurs this influence has to be considered by a factor from 0 to 3.

Factor 6:

Service temperature: If the service temperature is not below -10° C., or $+14^{\circ}$ F., the factor shall be zero. If temperature decreases, the factor shall be increased by one for each 5° C., or 2.8° F. Therefore, if the temperature is between -11° C., and -15° C., we shall have a value of 1, for -16° C. to -20° C. a value of 2, etc. The maximum value shall be 5.

Factor 7:

Structural integrity: Finally, we must take into account the importance of the particular structural part. It is evident that a failure in a main girder of a bridge is much more dangerous than a failure in the sidewalks or railings. Therefore for important parts, a failure of which would cause a collapse of the whole structure, an additional value of 5 is added.

All the numerical values may be graduated between zero and the maximum, depending upon the particular case.

By the addition of these factors we get a total danger-value which in general may range from ten to thirty. The steels corresponding to the standard DIN 17,100 have been characterized by quality values of

0 to 14 for grade 1
15 to 21 for grade 2
22 to 26 for grade 3

This quality-value must be higher or equal to the danger-value. If the danger-value is for instance 19, a steel of grade 2 is required. For a danger-value above 26, a special steel is necessary.

Professor Bierett has concentrated the various Factors in a few values, and assembled the values in a diagram. He has introduced a factor K, characterizing the severity of the conditions and the importance of a part; it comprises Factors No. 1, 2, 3 and 7 of the above recommendations. If the structure performs under very mild conditions, i.e., if the thickness of the plates is small and a failure would not be very dangerous, he proposes a factor of 0.5. If the structure performs under very severe conditions, he uses a factor of 2 as a maximum and between these limits factors of 0.7; 1.0; 1.4 are applied.

In his diagram (Figure 7) he introduces these K factors, the lowest service temperature, and the thickness. These three quantities define points in a field and this field contains zones, separated by graded lines for the different steel grades. We can immediately see the necessary steel grade. In a more complete diagram the influence of straightening procedure can be taken into consideration as well.

These recommendations were originally based only on practical experience. It was assumed that for severe conditions a finer steel is necessary - grade 3 according to DIN 17,100; for mild conditions a steel of grade number 1 would be sufficient. The limits for the application of these steels were estimated and have been checked with the ideas explained before, with all known test results, and with experience. For instance: If we allot one point in the danger value for each 5^{o} lowering of temperature, it is logical to consider also the influence of thickness and allot one point in danger value for an increase of thickness which will change the critical temperature by 5^{o} as well. Furthermore, the tests carried out on welded beams show the temperature for which the applied steel is appropriate. By means of direct comparison we may then determine whether the recommendations give the same result.

It is evident that many objections may be raised against this system. The first objection was, and there is much to say for it, that this system gives the impression of an accuracy which by no means is justified.

The second objection was just the opposite of the first one: The German industry said that the inaccuracy was too great, and therefore several engineers would get several different numbers for the danger-values. This objection could be checked. The German Committee for Structural Steel Work transmitted the details of structural parts to several firms and asked the engineers to apply the recommendations to calculate the danger-values and to ascertain the adequate steels. The result was satisfactory. Of course the calculated values were not exactly the same, but in nearly all cases the same grade of steel was selected. Only in those cases where the danger values were near the limits for the particular steel grade did differences occur.

A third and very important objection was that the background of the whole method is unsound. Since the existence of cracks cannot always be avoided, the propagation of such cracks, if one crack exists, has to be avoided. In this case the characteristics of the construction would be much less important than assumed in the recommendations.

All of these objections, as well as others, have been discussed in Commission IX of I.I.W. As a result Commission IX has set up a Sub-Commission to study the whole problem of the classification of structural parts. Furthermore, the attention of Commission XV has been drawn to the problem of classification and Mr. Guerrera, the chairman of Commission XV, has formed a working group to study the problem. Of course, a close collaboration is foreseen between these two groups, and we hope that in the future better proposals than those shown herein may be possible.

I believe the need for this work is justified by the following features which summarize the decisive points of my explanations:

(1) Since we have grades of steels which have the same mechanical properties (yield point, fracture strength, elongation and lateral contraction) and differ only by their liability to embrittlement, the engineer is compelled to choose an adequate steel for his structures by taking into account the question of embrittlement.

(2) Since the generally adopted criterion for the ductility of a test specimen is defined at a critical temperature or transition temperature, and since in structural parts the danger of brittle fracture occurs only if the service temperature is under a certain critical temperature--depending on the properties of the steel and the conditions of construction and loading--it is necessary to study the correlation between these two temperatures.

(3) As experts in this field we have a duty to help the many practicing engineers who are not familiar with all the research work, and to give them some guidance for the proper selection of steels. Even if such a recommendation is an approximate one, it is better than no recommendation at all.

We hope that in this very complicated and extended field of research the international cooperation will lead to interesting, fruitful, and essential results.

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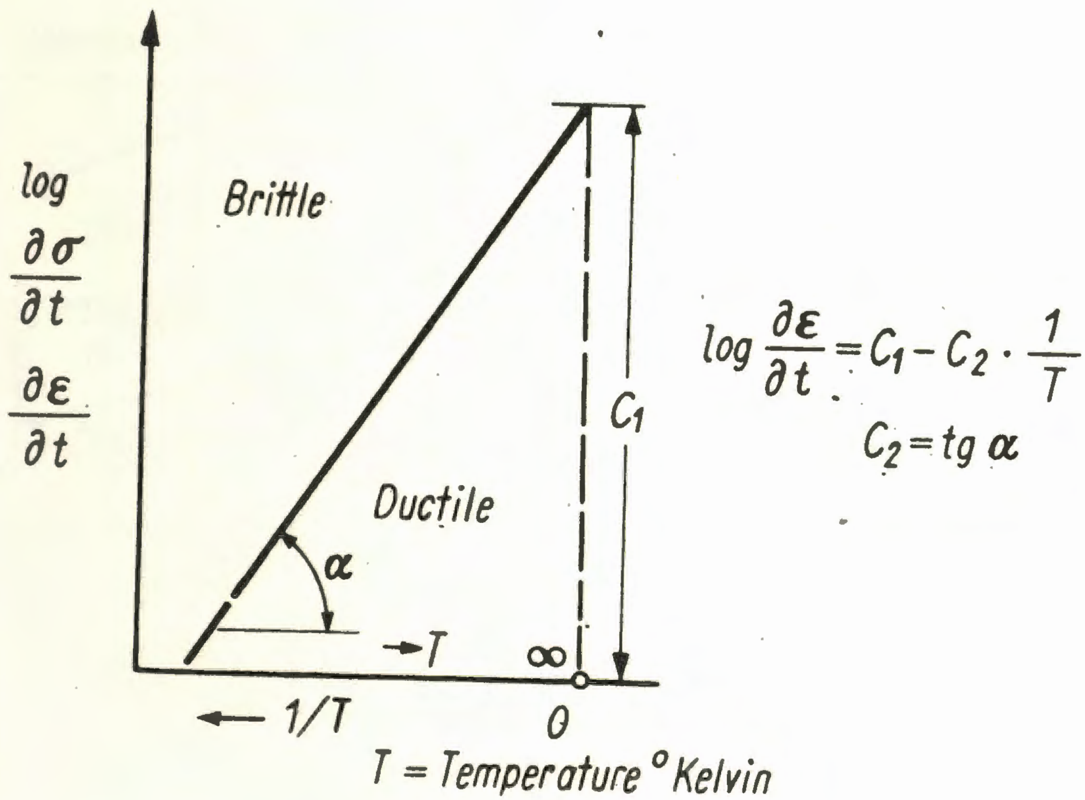

Figure 1 Correlation between strain rate and the temperature of embrittlement

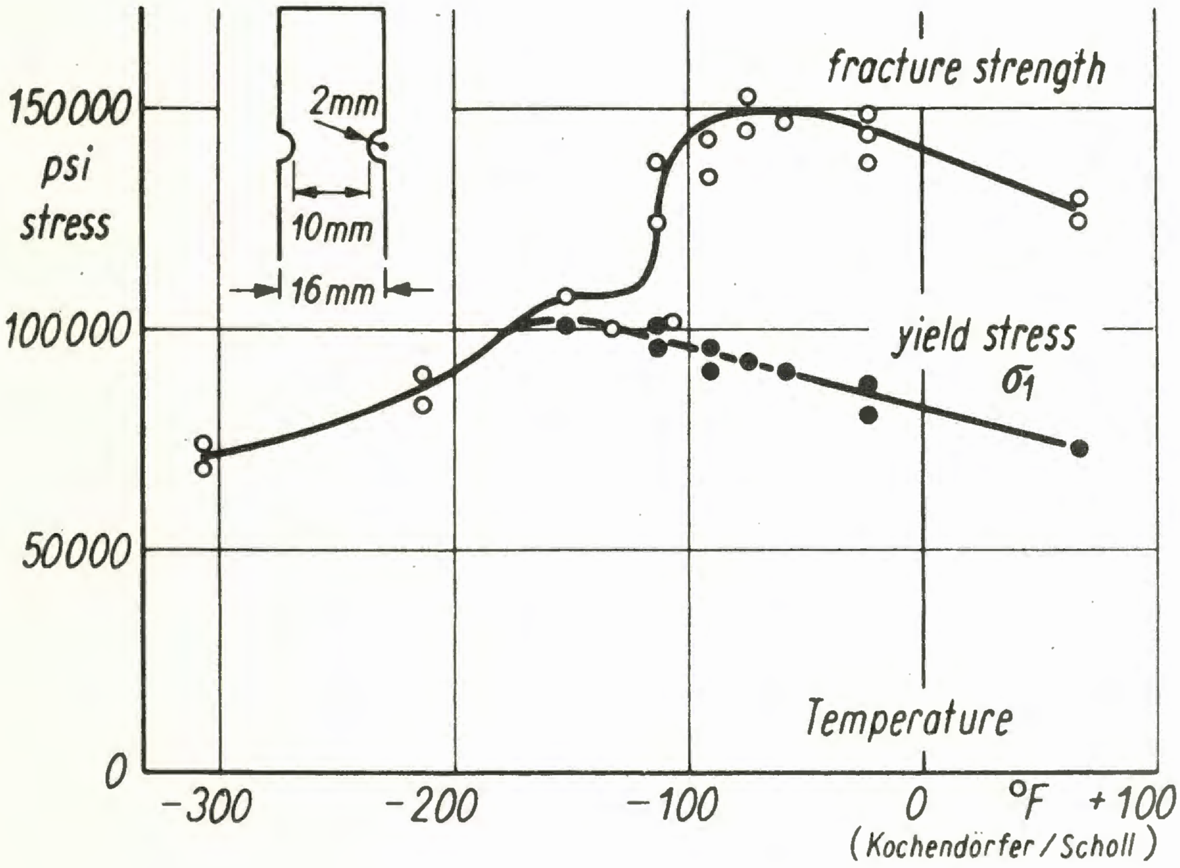

Figure 2 Fracture strength and yield stress as function of temperature, according to tests in the Max-Planck-Institute for Iron Research in Düsseldorf, Germany

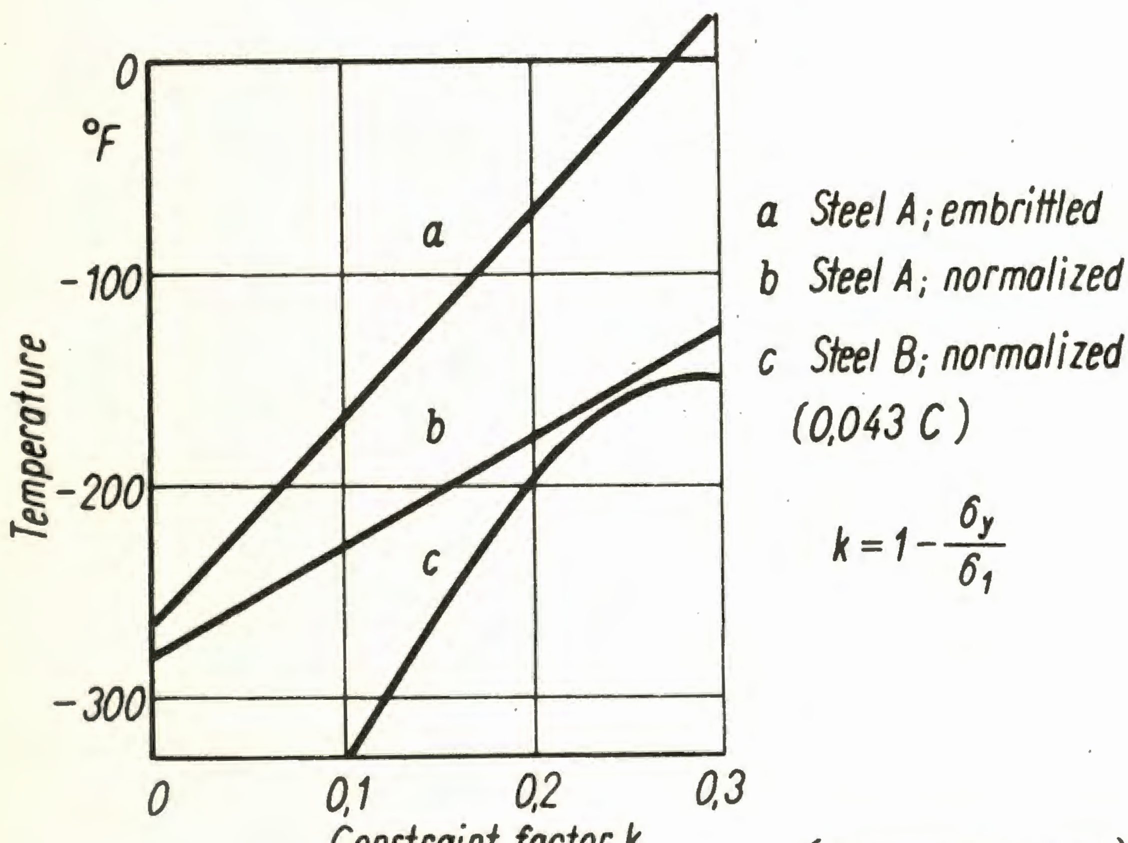

Figure 3 Transition Temperature as function of the constraint factor k; according to tests in the Max-Planck-Institute for Iron Research in Düsseldorf, Germany

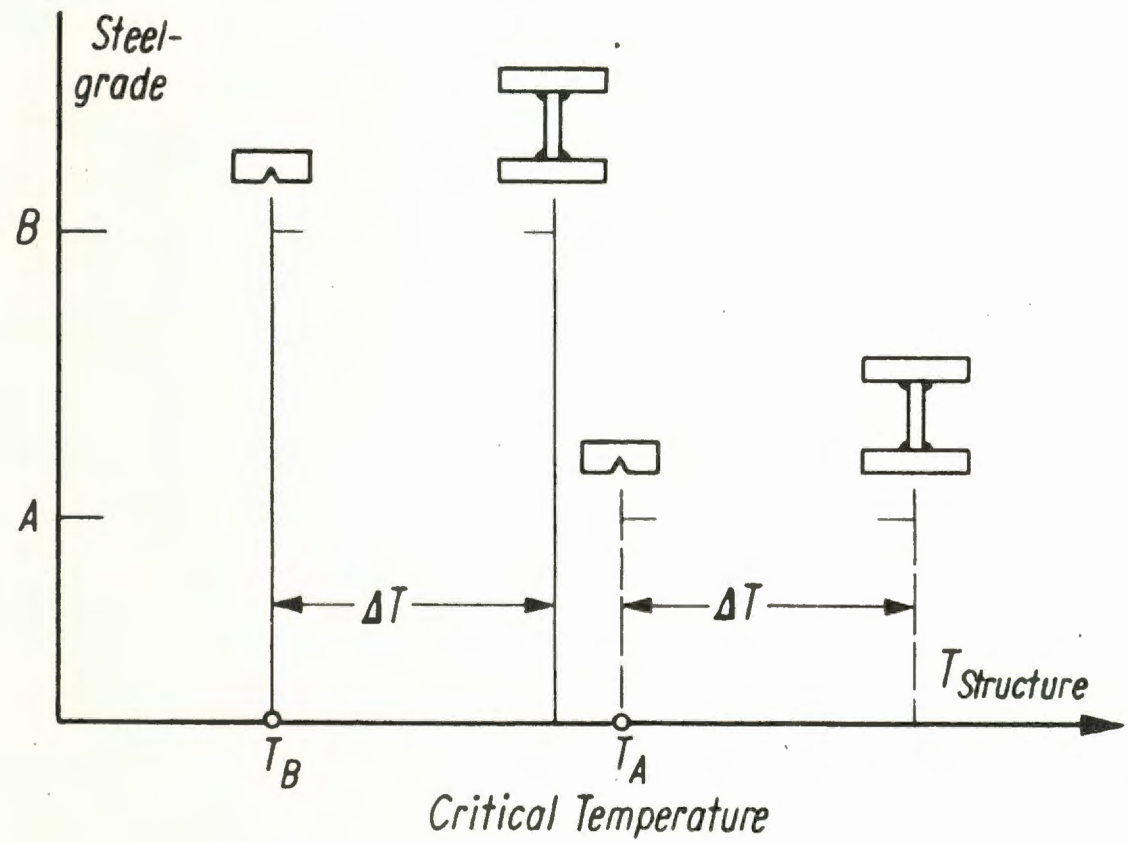

Figure 4 Critical temperatures of test specimens and structural parts (schematically)

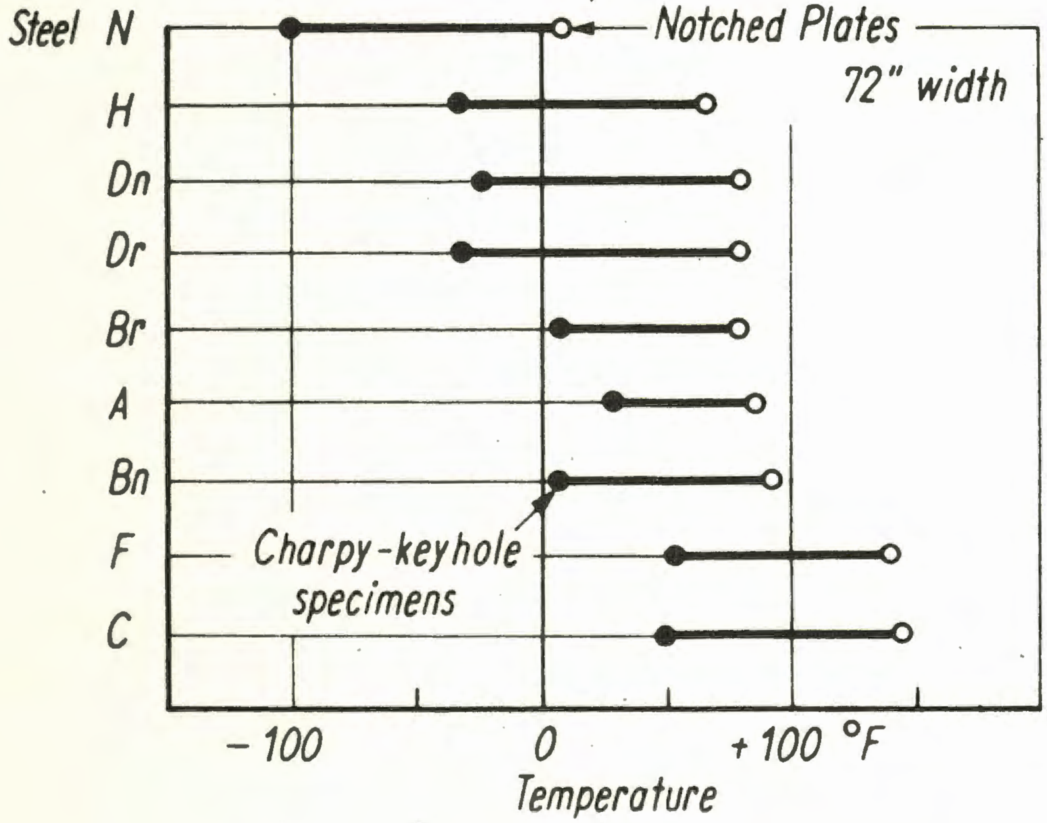

Figure 5 Critical temperatures of test specimens and wide notched plates (according to tests in U.S.A.)

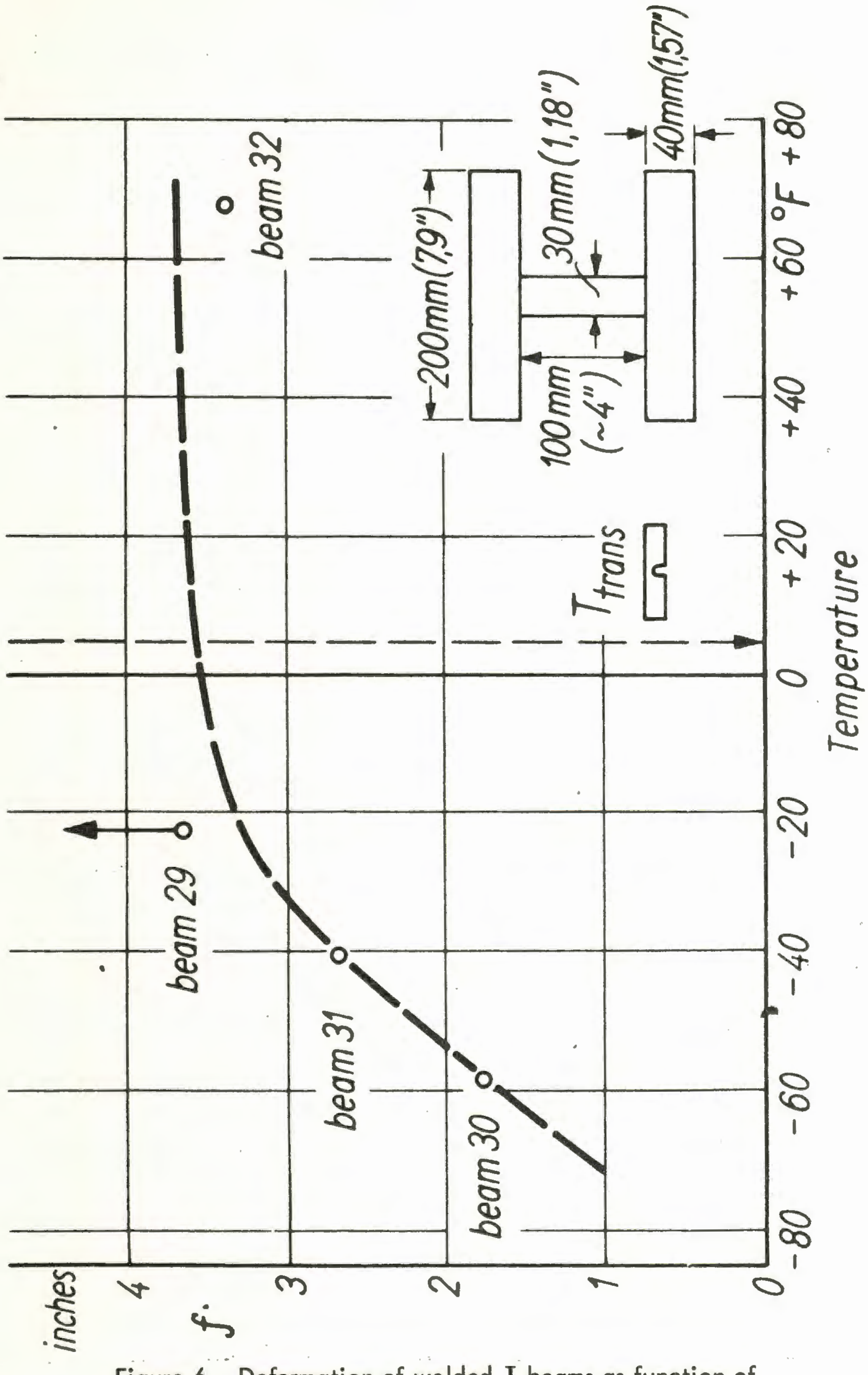

Figure 6 Deformation of welded I-beams as function of temperature, according to tests in the Federal Laboratories for Testing Materials in Berlin-Dahlem, Germany

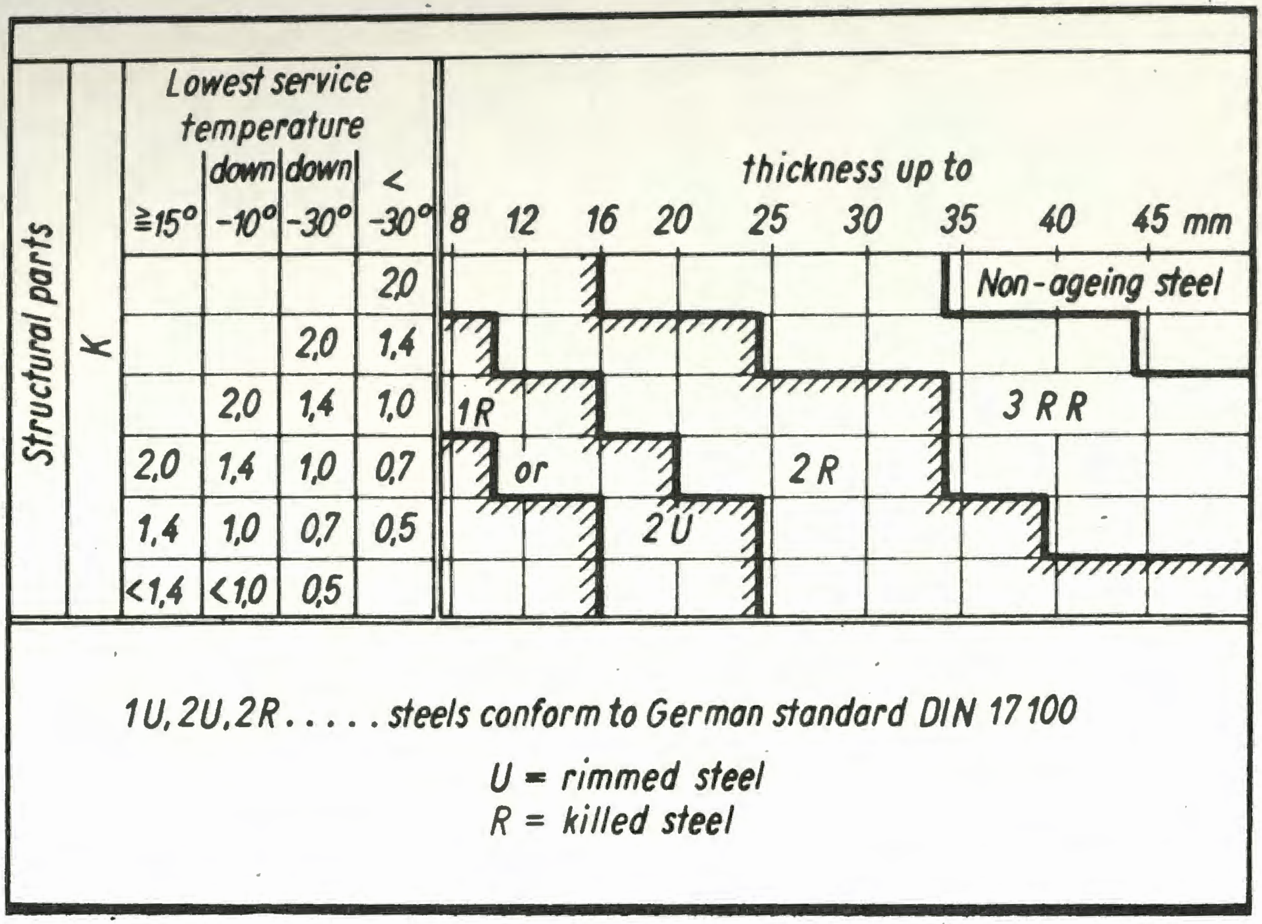

Figure 7 Chart for the choice of steel quality for welded constructions, according to proposals by Prof. Bierett

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